PROCEEDINGS

OF THE

AMERICAN SOCIETY

OF

CIVIL ENGINEERS

(INSTITUTED 1852)

VOL. XXXV—No. 1.

JANUARY, 1909.

Edited by the Secretary, under the direction of the Committee on Publications.

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NEW YORK 1909

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American Society of Civil Engineers

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The House of the Society is open from 9 A.M. to 10 P.M. every day, except Sundays, Fourth of July, Thanksgiving Day and Christmas Day.

HOUSE OF THE SOCIETY—220 WEST FIFTY-SEVENTH STREET, NEW YORK.

TELEPHONE NUMBER.....5913 Columbus.
CABLE ADDRESS....."Ceas, New York."

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Vol. XXXV. JANUARY, 1909. No. 1.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PROCEEDINGS

This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

SOCIETY AFFAIRS

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MINUTES OF MEETINGS

OF THE SOCIETY

December 16th, 1908.—The meeting was called to order at 8.30 P. M.; President Charles Macdonald in the chair; Chas. Warren Hunt, Secretary; and present, also, 148 members, and 29 guests.

A paper by William L. Sibert, M. Am. Soc. C. E., entitled "The Improvement of the Ohio River," was presented by the Secretary, who also read a communication on the subject from Theron M. Ripley, M. Am. Soc. C. E.

The paper was discussed orally by Messrs. G. W. Kittredge, R. B. Stanton, and Thomas C. Atwood.

The Secretary announced the following deaths:

EDMUND LOUIS DUBARRY, elected Associate, January 6th, 1875; died December 4th, 1908.

CHARLES TARBELL DUDLEY, elected Junior, September 6th, 1904; Associate Member, October 3d, 1906; died September 30th, 1908.

LESLIE MONROE FRY, elected Junior, June 6th, 1905; Associate Member, April 3d, 1907; died November 10th, 1908.

EDWARD PRINCE, elected Junior, February 6th, 1878; Member, November 1st, 1882; died December 12th, 1908.

Adjourned.

January 6th, 1909.—The meeting was called to order at 8.30 P. M.; President Charles Macdonald in the chair; Chas. Warren Hunt, Secretary; and present, also, 169 members, and 36 guests.

A paper by George B. Francis, M. Am. Soc. C. E., entitled "Electric Railways in the Ohio Valley Between Steubenville, Ohio, and Vanport, Pennsylvania," was presented by the author, and illustrated with lantern slides.

The paper was discussed by Messrs. F. Lavis, W. J. Boucher, George B. Preston, and the author.

The Secretary announced the election of the following candidates by the Board of Direction on January 5th, 1909:

AS ASSOCIATE MEMBERS.

SAINT GEORGE HENRY COOKE, Chester, Pa.
JOHN GEORGE LAWRENCE CUNNINGHAM, Williston, N. Dak.
DAVID LESLIE DIEHL, Harrisburg, Pa.
BYRON JAMES LAMBERT, Iowa City, Iowa.
EDWARD PERCY LANE, New York City.
HIRAM MILLER, La Paz, Bolivia.
MYRON HALL PECK, Tientsin, China.
FRANCIS WILLIAM PERRY, New York City.
EDWIN JOB PICKWICK, Schenectady, N. Y.
GEORGE ABEL PIERCE, Urbana, Ohio.
GEORGE SHIRLEY WALTER, Chicago, Ill.

AS JUNIORS.

GEORGE SIMPSON ARMSTRONG, Jr., Brooklyn, N. Y.
JOSEPH LAWRENCE BRENNAN, New York City.
HARRY COLLINS WALTON, New York City.

The Secretary announced the transfer of the following candidates by the Board of Direction on January 5th, 1909:

FROM ASSOCIATE MEMBER TO MEMBER.

HERMAN KARL ENDEMANN, Long Island City, N. Y.

FROM JUNIOR TO ASSOCIATE MEMBER.

ALFRED THOMAS BROWN, White Plains, N. Y.
HERBERT WILLARD GODDARD, Boston, Mass.

The Secretary announced the election of the following candidate by the Board of Direction on October 6th, 1908:

AS JUNIOR.

DAVID ADAMS CALHOUN, New York City.

The Secretary announced the following resignations as of December 31st, 1908:

Members: CHARLES MACRITCHIE, THOMAS DELANO WHISTLER.
Associate Members: AUSTIN CURTIN HARPER, CARL LOUIS EDUARD SCHENK.
Associate: WILLIAM FERRIS BOOTH.
Juniors: WILLIS ROBERT JORDAN, WILLIAM HENRY SNYDER.

The Secretary announced the following deaths:

RUTGER BLEECKER GREEN, elected Junior, May 5th, 1896; Associate Member, October 5th, 1898; Member, September 6th, 1904; died December 8th, 1908.

JOHN EGBERT MCCURDY, elected Member, April 1st, 1896; died December 15th, 1908.

Adjourned.

OF THE BOARD OF DIRECTION

(Abstract.)

January 5th, 1909.—President Macdonald in the chair; Chas. Warren Hunt, Secretary; and present, also, Messrs. Andrews, Churchill, Gibbs, Hazen, Schneider, Smith, Stearns, Swensson, and Tillson.

It was decided that the next Annual Convention of the Society shall be held at the Mount Washington Hotel, Bretton Woods, N. H., July 6th to 9th, 1909.

A report of the Board covering the year ending December 31st, 1908, for presentation to the Annual Meeting, was adopted.

Ballots for Membership were canvassed, resulting in the election of 11 Associate Members and 3 Juniors, and the transfer of 2 Juniors to the grade of Associate Member.*

The Secretary reported the entry of *Transactions* as second-class mail matter, beginning with Vol. LXI, for December, 1908.

Charles Macdonald was appointed to represent the Society on the John Fritz Medal Board of Award for four years.

Action was taken on members in arrears for dues.

The resignations of 2 Members, 2 Associate Members, 1 Associate, and 2 Juniors were received and accepted, taking effect as of December 31st, 1908.†

Applications were considered, and other routine business transacted.

Adjourned.

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* See page 2.
† See page 3.

ANNOUNCEMENTS

The House of the Society is open from 9 A. M. to 10 P. M., every day, except Sundays, Fourth of July, Thanksgiving Day, and Christmas Day.

MEETINGS

Wednesday, February 3d, 1909.—8.30 P. M.—A paper by Robert Spurr Weston, Assoc. M. Am. Soc. C. E., entitled "The Purification of Ground-Waters Containing Iron and Manganese," will be presented for discussion.

This paper is printed in *Proceedings* for December, 1908.

Wednesday, February 17th, 1909.—8.30 P. M.—At this meeting a paper by R. P. Bolton, M. Am. Soc. C. E., entitled "The Operation of Passenger Elevators," will be presented for discussion.

This paper is printed in *Proceedings* for December, 1908.

Wednesday, March 3d, 1909.—8.30 P. M.—Two papers will be presented for discussion, as follows: "The Action of Frost on Cement and Cement Mortar, Together with Other Experiments on These Materials," by Messrs. Ernest R. Matthews and James Watson; and "The Bonding of New to Old Concrete," by E. P. Goodrich, M. Am. Soc. C. E.

These papers are printed in this number of *Proceedings*.

ANNUAL CONVENTION

The Forty-first Annual Convention of the Society will be held at the Mount Washington Hotel, Bretton Woods, N. H., from July 6th to July 9th, 1909, inclusive.

PAPERS AND DISCUSSIONS

The Board of Direction has decided to issue *Transactions* in future as a quarterly publication. There will therefore be four volumes each year instead of two as heretofore.

There seems to be some misunderstanding about this matter, several members having protested on the ground that the present volumes are not too large, and that making them smaller would make reference to them more difficult, as well as increase the cost of binding.

It seems, therefore, necessary to explain that the decision of the Board is based on the fact that the amount of valuable material offered the Society for publication has so increased of late that a large part of it would have to be refused if only two volumes per annum were issued, and on the belief that the most effective way of increasing the usefulness of the Society to its Members and the Engineering Profession is to increase the volume of its Publications.

The decision was arrived at only after most careful consideration of the matter from every point of view, and it is hoped the result will be satisfactory to the membership.

It is also hoped that members and others who take part in the discussion of the papers presented will revise their remarks promptly, and that all written communications from those who cannot attend the meetings will be sent in at the earliest possible date after the issue of the paper in *Proceedings*. The issue of volumes of *Transactions* is dependent on the closing of discussions, and the co-operation of the membership will now be more necessary in this matter than heretofore, because four volumes are to be issued during the year instead of two, and, to accomplish this, a definite date of issue for each must be established. It is expected that the first volume for 1909 will be issued on or about March 31st, the second on June 30th, and the third and fourth on September 30th and December 31st, respectively.

All papers accepted by the Publication Committee are classified by the Committee with respect to their availability for discussion at meetings.

Papers, which, from their general nature, appear to be of a character suitable for oral discussion will be published as heretofore in *Proceedings*, and set down for presentation to a future meeting of the Society, and, on these, oral discussion, as well as written communications, will be solicited.

All papers which do not come under this heading, that is to say, those which, from their mathematical or technical nature, in the opinion of the Committee, are not adapted to oral discussion, will not be scheduled for presentation to any meeting. Such papers will be published in *Proceedings* in the same manner as those which are to be presented at meetings, but written discussions, only, will be requested for subsequent publication in *Proceedings* and with the paper in the volumes of *Transactions*.

PRIVILEGES OF ENGINEERING SOCIETIES EXTENDED TO MEMBERS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

Members of the American Society of Civil Engineers will be welcomed by the following Engineering Societies, both to the use of their Reading Rooms and at all Meetings:

American Institute of Mining Engineers, 29 West Thirty-ninth Street, New York City.

Associação dos Engenheiros Civis Portuguezes, Lisbon, Portugal.

Australasian Institute of Mining Engineers, Melbourne, Victoria, Australia.

Boston Society of Civil Engineers, 715 Tremont Temple, Boston, Mass.

Brooklyn Engineers' Club, 197 Montague Street, Brooklyn, N. Y.

Canadian Society of Civil Engineers, 877 Dorchester Street, Montreal, Que., Canada.

Civil Engineers' Club of Cleveland, 718 Caxton Building, Cleveland, Ohio.

Civil Engineers' Society of St. Paul, St. Paul, Minn.

Cleveland Institute of Engineers, Middlesbrough, England.

Engineers' and Architects' Club of Louisville, Ky., 303 Norton Building, Fourth and Jefferson Streets, Louisville, Ky.

Engineers' Club of Baltimore, Baltimore, Md.

Engineers' Club of Central Pennsylvania, Corner Second and Walnut Streets, Harrisburg, Pa.

Engineers' Club of Philadelphia, 1317 Spruce Street, Philadelphia, Pa.

Engineers' Club of St. Louis, 3817 Olive Street, St. Louis, Mo.

Engineers' Club of Toronto, 96 King Street West, Toronto, Ont., Canada.

Engineers' Society of Western Pennsylvania, 803 Fulton Building, Pittsburg, Pa.

Institute of Marine Engineers, 58 Romford Road, Stratford, London, E., England.

Institution of Engineers of the River Plate, Buenos Aires, Argentine Republic.

Institution of Naval Architects, 5 Adelphi Terrace, London, W. C., England.

Junior Institution of Engineers, 39 Victoria Street, Westminster, S. W., London, England.

Koninklijk Instituut van Ingenieurs, The Hague, The Netherlands.

Louisiana Engineering Society, 604 Tulane-Newcomb Building, New Orleans, La.

Memphis Engineering Society, Memphis, Tenn.

Midland Institute of Mining, Civil and Mechanical Engineers, Sheffield, England.

Montana Society of Engineers, Butte, Montana.

North of England Institute of Mining and Mechanical Engineers, Newcastle-upon-Tyne, England.

Oesterreichischer Ingenieur- und Architekten-Verein, Eschenbachgasse 9, Vienna, Austria.

Pacific Northwest Society of Engineers, 617-618 Pioneer Building, Seattle, Wash.

Rochester Engineering Society, Rochester, N. Y.

Sachsischer Ingenieur- und Architekten-Verein, Dresden, Germany.

Sociedad Colombiana de Ingenieros, Bogota, Colombia.

Societe des Ingenieurs Civils de France, 19 Rue Blanche, Paris, France.

Society of Engineers, 17 Victoria Street, Westminster, S. W., England.

Svenska Teknologföreningen, Brunkebergstorg 18, Stockholm, Sweden.

Tekniske Forening, Vestre Boulevard 18–1, Copenhagen, Denmark.

Western Society of Engineers, 1737 Monadnock Block, Chicago, Ill.

SEARCHES IN THE LIBRARY

In January, 1902, the Secretary was authorized to make searches in the Library, upon request, and to charge therefor the actual cost to the Society for the extra work required. Since that time many searches have been made, and bibliographies and other information on special subjects furnished.

The resulting satisfaction, to the members who have made use of the resources of the Society in this manner, has been expressed frequently, and leaves little doubt that, if it were generally known to the membership that such work would be undertaken, many would avail themselves of it.

The cost is trifling compared with the value of the time of an engineer who looks up such matters himself, and the work can be performed quite as well, and much more quickly, by persons familiar with the Library.

In asking that such work be undertaken, members should specify clearly the subject to be covered, and whether references to general books only are desired, or whether a complete bibliography, involving search through periodical literature, is desired.

In reference to this work the Appendix* to the Annual Report of the Board of Direction for the year ending December 31st, 1906, contains a summary of all searches made to that date.

* *Proceedings*, Vol. XXXIII, p. 20 (January, 1907).

ANNUAL REPORT OF THE BOARD OF DIRECTION FOR THE YEAR ENDING DECEMBER 31ST, 1908.

In compliance with the Constitution, the Board of Direction presents its report for the year ending December 31st, 1908.

MEMBERSHIP.

The changes in membership are shown in the following table:

Grade.	Jan. 1st, 1908.			Jan. 1st, 1909.			Losses.				Additions.		Totals.	
	Resident.	Non-Resident.	Total.	Resident.	Non-Resident.	Total.	Transfer.	Resignation.	Dropped.	Death.	Transfer.	Election.	Loss.	Gain.
Honorary Members	.....	8	8	.....	8	8	....	..	..	..	....	....	....	....
Corresponding Members	.....	2	2	.....	2	2	....	..	..	..	....	....	....	....
Members	492	1 726	2 218	525	1 845	2 370	....	4	5	35	*79	117	44	196
Associate Members	385	1 056	1 441	401	1 218	1 619	76	4	6	6	†64	206	92	270
Associates	62	94	156	68	93	161	3	3	2	2	....	15	10	15
Juniors	137	426	563	152	490	642	64	5	7	1	....	156	77	156
Fellows	9	14	23	7	15	22	....	..	..	1	....	....	1	....
Total	1 085	3 326	4 411	1 153	3 671	4 824	143	16	20	45	143	494	224	637

* 76 Associate Members, 1 Associate, and 2 Juniors.
† 2 Associates and 62 Juniors.

The net increase during the year, 413, while not as great as that of 1907, doubtless due to the prevailing financial conditions, is much greater than the increase in any other previous year. It would have been somewhat larger had it not been for the amendment to the Constitution which went into effect in November, and to the changed rules of the Board of Direction, which deferred the election of the candidates on one preliminary list until after December 31st. The average net increase for the past five years has been 380.

The number of applications received during the year was 728: 536 for admission, and 192 for transfer.

The losses by death reported during the year number 45, and are as follows:

Members (35): George Irving Bailey, Oliver Weldon Barnes, Elbridge Harlow Beckler, Walter Gilman Berg, William Beverly Chase, James Dun, John Edwin Earley, Nathaniel Marsh Edwards, George Edwin Evans, Irving Tupper Farnham, Rutger Bleecker Green, John Bradford Harper, Charles Hermany, Wilhelm Hildenbrand, Henry Randolph Holbrook, John Egbert McCurdy, Martin William Mansfield, Aniceto G. Menocal, Thomas C. Meyer, Stephen Arnold

Mitchell, Othniel Foster Nichols, Herbert Franklin Northrup, William Anson Pearson, Jr., Frederic Auten Combs Perrine, Edward Prince, George W. Rafter, Louis Younglove Schermerhorn, Mark William Schofield, George Edward Sleeper, Edward Clinton Terry, George Edward Thomas, Richard Fenwick Thorp, Charles Harold Tutton, Clarence George Vaughn, Frederick Conover Warman.

Associate Members (6): Charles Tarbell Dudley, Leslie Monroe Fry, James Isaac Haycroft, William Seaton, Jr., Joseph Milton Walker, Clinton Glencairn Wells.

Associates (2): Edmund Louis DuBarry, William Roberts.

Juniors (1): Adolph DeHaas.

Fellows (1): Nathaniel S. Bouton.

LIBRARY.

The total contents of the Library, and the increase during the year, are shown in the following statement:

	Total Contents.	Increase during 1908.
Bound volumes	17 590	1 161
Unbound volumes	35 300	3 564
Specifications	6 633	46
Maps, photographs and drawings	4 090	209
Total	63 613	4 980

Of these accessions, 2 074 were donations received in answer to special requests; 86 were donations from publishers; 2 672 were donations received in regular course, and 148 were purchased.

The value of accessions to the Library during the year is as follows, each accession having been valued separately as received:

4 832 Donations and exchanges (estimated value)	$2 585.21
148 Volumes purchased (cost)	389.63
Binding 318 volumes	407.89
Total	$3 382.73

The following amounts have been expended upon the Library during the year:

Purchase of books, subscriptions, express charges, etc.	$416.29
Binding	407.89
Fixtures, supplies, and sundries	100.83
Total	$925.01

The number of titles in the Library is 23 408.

The total attendance in the Reading Room and Library during the year was 4 285; a considerable increase over previous years.

During the year 74 new bibliographies (containing 2 253 separate references) have been made, copies of 30 searches made in previous years have been furnished, 7 of these having been brought up to date. The total cost of this work, $473.10, has been paid by those for whom it was undertaken.

In November, 1897, the Society moved into the present House, and on December 31st, 1897, the total number of books, pamphlets, specifications, maps, etc., in the Library was about 27 000. In the report of the Board of Direction for that year, attention was called to the general condition of the Library, and the hope was expressed that the new conditions would result in large and valuable additions. Attention was also called to the amount of shelf room provided in the New House, of which it was estimated the contents of the Library would then occupy about one-third. As stated above, the number of accessions has increased until at the present time it is 63 613, and the vacant shelf room referred to eleven years ago will soon be insufficient to take care of the normal growth. In consequence of this, an additional tier of stacks, space for which was wisely provided at that time, has been ordered, and will be installed early in 1909. This will double the present capacity, and will probably be sufficient to take care of the accessions of the next ten or fifteen years. Owing to the enlargement of the House in 1905, there is also available for future use additional space, which will admit of further extension of the shelving whenever it is needed.

PUBLICATIONS.

During the year ten numbers of *Proceedings* have been issued regularly, and two volumes of *Transactions* have appeared.

In *Proceedings,* the list of references to current engineering literature has been continued, and has covered 81 pages, and contained 3 297 classified references to periodicals.

The stock of the various publications of the Society kept on hand for the convenience of members and others now amounts to 143 626 copies, the cost of which to the Society for paper and press work only has been $19 745.14.

In addition to the regular publications, the second volume of the Index to *Transactions,* covering Volumes XLVI to LIX (20 volumes, including the six extra Engineering Congress volumes), was compiled during the year and issued to the membership without extra charge.

During the year, 8 356 Volumes of *Transactions,* and 2 737 copies of the Index to *Transactions,* have been bound for members and others in the standard half-morocco and cloth bindings.

SUMMARY OF PUBLICATIONS FOR 1908.

	Issues.	Average Edition.	Total Pages.	Plates.	Cuts.
Transactions (Volumes).....	2	5 075	1 209	131	233
Proceedings (Monthly Numbers).	10	5 295	1 988	145	268
Constitution and List of Members	1	5 500	346	...	1
Index to *Transactions,* Vols. XLVI to LIX.....	1	5 200	110	...	...
Totals.....	14	....	3 653	276	502

The cost of publications has been:

For Paper, Printing, Binding, etc., *Transactions* and *Proceedings*	$17 769.87
For Plates and Cuts.....	2 589.94
For Boxes, Mailing Lists, Copyright and Sundry Expenses.	795.35
For 8 000 Extra Copies of Memoirs and Papers.....	1 048.68
For List of Members.....	1 646.50
For Index to *Transactions,* Vols. XLVI to LIX.....	462.69
Total.....	$24 313.03
Deduct amount received from sale of publications.....	3 268.95
Net cost of publications for 1908.....	$21 044.08

A comparison of these figures with those given in the last report of the Board shows that in *Transactions* and *Proceedings* 315 more pages were issued in 1908 than in 1907. This indicates very clearly that the amount of material offered the Society for publication has increased very much of late, and your Board has decided to enlarge the publications. In order to do this, it will be necessary to issue more than two volumes per annum, because a volume of more than 600 pages is unwieldy, unsightly and inexpedient. The *Transactions,* therefore, will be issued hereafter as a quarterly, which form of publication enables the Society to effect a considerable saving in the matter of postage. It is the belief of your Board that the most effective manner to increase the usefulness of the Society to its members is by the enlargement of its technical publications, and that, while the additional expense will be great, the financial condition of the Society warrants the expenditure, and it is hoped that members will do their part by furnishing original papers, and discussing those written by others.

MEETINGS.

During the year, 25 meetings have been held as follows: At the Annual Meeting, 2; at the Annual Convention, 5; and 18 other meetings held at the Society House.

At these meetings there were presented 17 formal papers, 8 of which were illustrated with lantern slides, besides which there were 4 topical discussions and 3 lectures, 2 of which were illustrated. There were also 7 papers published in *Proceedings* which were not presented at any meeting of the Society. The number of members and others who took part in the preparation of, or discussion of, these papers was 220.

The Fortieth Annual Convention was held at Denver, Colo., and the meetings, excursions and entertainments were well attended, instructive, and enjoyable.

The total attendance at the twenty-five meetings held was about 5 050. The registered attendance at the Annual Meeting was 653, and at the Annual Convention 438 (includes members only), but there were many members and guests present at each of these meetings who failed to register.

MEDALS AND PRIZES.

For the year ending with the month of July, 1907, prizes were awarded as follows:

The Norman Medal to Leonard M. Cox, M. Am. Soc. C. E., for his paper entitled "The Naval Floating Dock—Its Advantages, Design and Construction."

The Thomas Fitch Rowland Prize to James D. Schuyler, M. Am. Soc. C. E., for his paper entitled, "Recent Practice in Hydraulic-Fill Dam Construction."

FINANCES.

During the year the mortgage debt has been reduced by a payment of $10 000, and now amounts to $155 000.

The attention of members is invited to the Secretary's statement of receipts and disbursements, and to the general balance sheet which accompanies it, in which the very satisfactory financial condition of the Society appears.

The reports of the Secretary and Treasurer are appended.

By order of the Board of Direction,

CHAS. WARREN HUNT,
Secretary.

NEW YORK, JANUARY 5TH, 1909.

REPORT OF THE SECRETARY FOR THE

To the Board of Direction of the

Gentlemen:—I have the honor to present a statement of Receipts 31st, 1908. I also append a general balance sheet showing the condition

Receipts.

Balance on hand, December 31st, 1907, in Bank, Trust Company, and in hands of Treasurer		$20 749.28
Entrance Fees	$11 869.55	
Current Dues	53 266.64	
Past Dues	1 570.40	
Advance Dues	23 407.40	
Compounding Dues	500.00	
Certificates of Membership	465.75	
Badges	2 654.88	
Sales of Publications	3 268.95	
Interest	610.15	
Library	496.54	
Convention	3.00	
Annual Meeting	1 272.50	
Binding	6 107.09	
Miscellaneous	139.65	
		105 632.50
		$126 381.78

YEAR ENDING DECEMBER 31st, 1908.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

and Disbursements for the fiscal year of the Society, ending December
of the affairs of the Society.

Respectfully submitted,
CHAS. WARREN HUNT,
Secretary.

DISBURSEMENTS.

Salaries of Officers	$11 533.31	
Clerical Help	14 436.86	
Caretaking	1 728.71	
Publications	24 313.03	
Library	824.18	
Library Maintenance	100.83	
Maintenance of House	177.08	
Heat, Light and Water	1 194.47	
Convention	1 309.80	
Annual Meeting	1 792.60	
Postage	6 857.26	
General Printing and Stationery	2 900.92	
Badges	2 246.50	
Certificates of Membership	337.60	
Prizes	120.00	
Binding	5 772.93	
Furniture	235.26	
Interest and Insurance	6 257.08	
Members' Accounts	48.80	
Current Business	957.53	
Petty Expenses	154.43	
Bond and Mortgage (Payment on Principal)	10 000.00	
		$93 299.18
Balance on hand, December 31st, 1908:		
In Union Trust Company	$18 470.69	
In Garfield National Bank	13 111.91	
In hands of Treasurer	1 500.00	
		33 082.60
		$126 381.78

GENERAL BALANCE SHEET, DECEMBER 31ST, 1908.

ACCOMPANYING THE REPORT OF THE SECRETARY.

ASSETS.		
Three Lots (estimated value)		$270 000.00
Society Building (cost)		166 240.73
Furniture (cost)		18 165.82
Publications on hand (inventoried cost value)		19 745.14
Library:		
Cash expended for books, etc.	$13 030.44	
Donations, estimated value	53 687.84	
		66 718.28
Due from Members	$3 609.05	
Due from Non-Members	830.31	
		4 439.36
Cash		33 082.60
		$578 391.93

LIABILITIES.		
Dues for 1909 paid in advance		$ 23 407.40
Mortgage Debt and Loan		155 000.00
Funds invested in Society House, Lots and Library.	$ 26 730.78	
Surplus	373 253.75	
		399 984.53
		$578 391.93

We have examined the books and accounts of the American Society of Civil Engineers, for the year ended December 31, 1908, and certify that the foregoing Balance Sheet is in accordance therewith, and, in our opinion, correctly states the condition of the Society's affairs, as shown by the books.

79 WALL STREET, NEW YORK.
JANUARY 14, 1909.

MARWICK, MITCHELL & CO.,
Chartered Accountants.

REPORT OF THE TREASURER.

In compliance with the provisions of the Constitution, I have the honor to present the following report for the year ending December 31st, 1908:

Balance on hand December 31st, 1907		$20 749.28
Receipts from current sources, January 1st to December 31st, 1908		105 632.50
Payment of Audited Vouchers for Current Business, January 1st to December 31st, 1908	$83 299.18	
Payment on principal of bond and mortgage	10 000.00	
Balance on hand December 31st, 1908:		
In Union Trust Company	$18 470.69	
In Garfield National Bank	13 111.91	
In hands of the Treasurer	1 500.00	
	33 082.60	
	$126 381.78	$126 381.78

Respectfully submitted,

JOS. M. KNAP,
Treasurer, Am. Soc. C. E.

NEW YORK, JANUARY 5TH, 1909.

GENERAL BALANCE SHEET, DECEMBER 31st, 1908.

ACCESSIONS TO THE LIBRARY

(From December 9th, 1908, to January 12th, 1909)

DONATIONS*

AIR LIQUIDE OXYGÈNE AZOTE.

Par Georges Claude. Préface de M. d'Arsonval. Paper, 10 x 6½ in., illus., 399 pp. Paris, H. Dunod et E. Pinat, 1909. 15 francs.

The author, it is stated, after an investigation of the subject, has become convinced of the necessity of a work on the liquefaction and separation of air into its elements such as is presented in this volume. The Contents are: Part I, La Liquéfaction des Gaz; Part II, La Liquéfaction Industrielle de l'Air; Part III, Conservation et Propriétés de l'Air Liquide; Part IV, La Séparation de l'Air en Ses Eléments.

RESERVOIRS FOR IRRIGATION, WATER-POWER AND DOMESTIC WATER-SUPPLY.

With an Account of Various Types of Dams and the Methods, Plans and Cost of Their Construction. By James Dix Schuyler, M. Am. Soc. C. E. Second Edition, Revised and Enlarged. Cloth, 10 x 7 in., illus., 26 + 573 pp. New York, John Wiley & Sons; London, Chapman & Hall, Limited, 1908. $6.00.

The first edition of this book was issued in 1901; since then, it is stated, there has been such an increase in the number of dams and reservoirs constructed and such a demand for information regarding their dimensions, character, plans, materials, methods of construction, and cost, that the author feels justified in bringing out this, the second edition. It is stated that much new material has been added to the subject-matter, and that some which has become obsolete has been omitted. The chapter on Hydraulic-Fill Dams has been greatly enlarged by descriptions of late constructions of this type. Chapter III, on Masonry Dams, has also been enlarged, mention being made of the most notable dams in the world of this type, as well as of many of the smaller ones. Plates 1, 2 and 3, show profiles of all the better known masonry dams in the world, drawn to uniform scale for easy graphical comparison. Chapters have also been added to the sections on reinforced concrete and structural steel dams, giving the latest practice in these types. According to a secondary title, the book also contains miscellaneous data on the available water supply for irrigation in various sections of Arid America; distribution, application, and use of water; rainfall and run-off from various water-sheds; evaporation from reservoirs; effect of silt on the useful life of reservoirs; average cost of reservoirs per unit of capacity, etc. The Contents are: Rock-Fill Dams; Hydraulic-Fill Dams; Masonry Dams; Earthen Dams; Steel Dams; Reinforced Concrete Dams; Natural Reservoirs; Miscellaneous. There is an Appendix containing tabulated data of the cost of reservoir construction per acre-foot in the United States and in foreign countries on various types of dams, also tables of area and capacity of twelve Western reservoirs, at varying levels.

MORRISON'S SPRING TABLES.

A Handbook for Engineers, Students, and Draughtsmen. By Egbert R. Morrison. Cloth, 9 x 6 in., 84 pp. Sharon, Pa., E. R. Morrison, 1908.

In his book the author has divided springs into two classes, namely, light and heavy, called wire and bar in the case of helical springs and sheet and plate in the case of elliptical springs. The basis of calculation in helical springs is said to have been taken as the ratio between the diameter of the bar and the mean diameter of the springs; in elliptical springs, it is the ratio between the thickness of the plate and the span or net length of the spring. The author states that in the tables he has arranged the properties of light springs under graduated values of the fundamental ratio, and those of heavy springs under each size bar or plate. The working base, in the case of helical springs, is stated to be 1 in. of solid height, that for elliptical springs being taken as one plate 1 in. wide. All calcula-

*Unless otherwise specified, books in this list have been donated by the publisher.

tions are said to be based on a fiber strain of 80 000 lb. per sq. in. The modulus of elasticity for helical springs is taken at 12 600 000, and for elliptical springs at 25 400 000. The loads are given in pounds, and all dimensions are in inches. The Contents are: Part I, Formulas; Part II, Mathematical Tables Supplementary to Formulas; Part III, Spring Tables.

THE VENTILATION OF PUBLIC SEWERS.

By John S. Brodie. Cloth, 9 x 6 in., illus., 170 pp. London, The St. Bride's Press, Limited, 1908. $2.40.

The author states that he has considered the methods of sewer ventilation under three heads, namely, ventilation by natural air currents; ventilation by artificially produced air currents; and deodorization of foul sewer air or gas before it is discharged into the open air. Under each head he describes the methods used in various cities and towns in England, the laws governing their use, and gives abstracts of reports and papers by engineers and engineering societies on the subject. Chapter VI is devoted to a discussion of comparative costs of modern sewer systems. The Chapter headings are: Introductory; Is the Ventilation of Sewers Necessary?; Ventilation by Natural Air Currents; Sewer Ventilation by Artificially-Produced Air Currents; Deodorisation of Sewer Air or Sewer Gas; Comparative Costs of the Various Systems now in Use; Summary and Conclusions. There is an Appendix which contains abstracts of reports, papers, etc., on sewer ventilation, and an index of four pages.

HENDRICKS' COMMERCIAL REGISTER OF THE UNITED STATES

For Buyers and Sellers. Especially Devoted to the Interests of the Architectural, Mechanical, Engineering, Contracting, Electrical, Railroad, Iron, Steel, Hardware, Mining, Mill, Quarrying, Exporting and Kindred Industries. Seventeenth Annual Edition. Cloth, 10 x 8 in., illus., 82 + 1240 pp. New York, Samuel E Hendricks Co., 1908. $10.00.

This is an annual index of the above-mentioned industries, and is said to contain more than 350 000 names and addresses of manufacturers and firms and upwards of 30 000 business classifications. Full lists of manufacturers of, and dealers in, everything used in the manufacture of materials, machinery, and apparatus for these industries are given. The contents are classified by subject, under which are given, in alphabetical order, and in some cases by States, the names and addresses of firms dealing in that particular article. There is an index of eighty-two pages.

LABORATORY NOTES ON INDUSTRIAL WATER ANALYSIS.

A Survey Course for Engineers. By Ellen H. Richards. Cloth, 9 x 6 in., 49 pp. New York, John Wiley & Sons; London, Chapman & Hall, Limited, 1908. 50 cents.

In the Introduction it is stated that for manufacturing purposes there are special requirements to be considered under each case relating to the water to be used for such purpose, and that the engineer, in deciding upon recommendations, is often required to estimate the value of such water. In this short course on industrial water analysis only these special methods are considered, it is stated. the ordinary analytical processes being omitted entirely. Part I of the book consists of five laboratory exercises. In Part II, the Contents are: Standard Solutions; Computation of Hypothetical Combinations; Percentage Composition of Salinity in Various Waters; Tables; Convenient Data; Some Useful References. This last chapter contains a short bibliography of the subject.

THE MODERN ASPHALT PAVEMENT.

By Clifford Richardson, M. Am. Soc. C. E. Second Edition, Revised and Enlarged. Cloth, 9 x 6 in., illus., 9 + 629 pp. New York, John Wiley & Sons; London, Chapman & Hall, Limited, 1908. $3.00.

The first edition of this work was published in 1905. Since that time there have been considerable developments in the industry, which have caused a com-

plete revision of the subject-matter in order to bring it up to date. The book includes, it is stated, descriptions of the forms of construction which have been found by engineers to be the most satisfactory; the character of the materials entering into the composition of asphalt pavements; the best methods of construction used at the present time, together with the reasons which led to their adoption; the proper methods of maintenance and the causes of their deterioration; and specifications for asphalt pavements to meet various uses. At the end of each chapter a brief résumé of the contents of that chapter is given. The Contents are: Part I, The Foundation and Intermediate Course; Part II, The Materials Constituting the Asphalt Surface Mixture; Part III, Native Bitumens in Use in the Paving Industry; Part IV, Technology of the Paving Industry; Part V, Handling of Binder and Surface Mixture on the Street; Part VI, The Physical Properties of Asphalt Surfaces; Part VII, Specifications for and Merits of Asphalt Pavements; Part VIII, Causes of the Defects in and the Deterioration of Asphalt Surfaces; Part IX, Control of Work.

A STUDY OF ORE DEPOSITS FOR THE PRACTICAL MINER.

With Descriptions of Ore Minerals, Rock Minerals, and Rocks. By J. P. Wallace. Cloth, 9½ x 6½ in., illus., 15 + 349 pp. New York and London, Hill Publishing Company, 1908. $3.00.

This book, it is stated, is a condensation of personal experiences covering a period of thirty years, the borrowed experience of practical mining men, and of facts and theories from all the available mining literature. It is said to be written for the miner, the prospector, and the mining public. A brief description of the most important minerals, ores, and rocks is given. The structural features of ore deposits and the walls enclosing them, together with the form, origin, and manner of occurrence of deposits, it is stated, have been given special attention. Descriptions of prominent mines of various types and forms, especially extensively developed properties with a history, are also given, together with a brief mention of the geology of each region. At the back of the book there is a list of the papers and publications which have been consulted. The Contents are: Part I, The Ore Minerals; Part II, the Rock Forming Minerals, Rocks and Rock Displacements; Part III, General Characters and Classes of Ore Deposits; Part IV, Some Types of Ore Deposits; Part V, Mine Valuation and Prospecting.

THE STEAM TURBINE.

A Practical and Theoretical Treatise for Engineers and Designers, Including a Discussion of the Gas Turbine. By James Ambrose Moyer. Cloth, 9 x 6 in., illus., 7 + 370 pp. New York, John Wiley & Sons; London, Chapman & Hall, Limited, 1908. $4.00.

It has been the intention of the author to make this book a manual for the practical engineer for use in the design, operation, or manufacture of steam turbines, and to explain in a general way the more important problems necessary to the qualified steam engineer. He takes up first the more simple problems of nozzle design, which he follows with a discussion of blade design and the steam turbine generally. At the back of the book there is an entropy-total heat chart which is laid out with lines of constant superheat instead of lines of constant temperature, which are generally used for charts of this kind. The Contents are: Introduction, The Elementary Theory of Heat; Nozzle Design; Steam Turbine Types and Blade Design; Mechanical Losses in Turbines; Method for Correcting Steam Turbine Tests; Commercial Types of Turbines; Governing Steam Turbines; Low-Pressure (Exhaust) Turbines; Marine Turbines; Tests of Turbines; Steam Turbine Economics; Stresses in Rings, Drums, and Disks; Gas Turbines; Electric Generators for Turbines. There is an index of eight pages.

ELEVATOR SERVICE.

By Reginald Pelham Bolton, M. Am. Soc. C. E. Cloth, 11 x 8 in., illus., 69 pp. New York, Reginald Pelham Bolton, 527 Fifth Avenue, 1908. $5.00.

In a secondary title, it is stated that the book contains descriptions of operating conditions and proportions, with diagrams, formulas, and tables for passenger travel, schedule and express operation, with the relation of the elevators to the building, and proportions and loads of cars. The Contents are: The Problem of Vertical Transportation; Operating Conditions; Passengers and Operators; Rating

the Work of the Elevator; Computing the Average Work; Express Service; The Shape and Size of the Car; Load and Speed Combinations; The Building and Its Proportionate Service; Examples of the Use of Figure XI; Definitions of Some Terms Used in Connection with Elevators.

STATICS BY ALGEBRAIC AND GRAPHIC METHODS.

Intended Primarily for Students of Engineering and Architecture. By Lewis Jerome Johnson, M. Am. Soc. C. E. Second Edition, Revised and Enlarged. Cloth, 9 x 6 in., illus., 11 + 169 pp. New York, John Wiley & Sons; London, Chapman & Hall, Limited, 1908. $2.00.

Several specific purposes have been attempted in this book, it is stated, to help students of engineering and architecture to acquire a knowledge of statics which will include the power to apply that knowledge correctly in practice. These purposes include: Attention to the starting points of science and clearness as to the course of deduction therefrom; the pointing out of the inherent mathematical limitations of pure statics and showing how all its important problems may be solved; the development of algebraic and graphic methods of solution side by side and with equal thoroughness; the presentation of a graded set of problems illustrating not only universal principles, but also how statics is used in engineering practice, etc. The most important additions to the subject-matter, in this edition, will be found, it is stated, mostly in the appendices, and consist of a detailed and time-tested scheme for assigning individual data for the exercises in the text and a collection of practice problems with answers, which have been collected from various sources. There are also a short syllabus and seven new double-paged plates which give solutions of certain of the exercises. The Contents are: Part I, General Principles and Methods: Introduction; Definitions and Preliminaries; Notation and Conventions; Parallelogram of Forces and Its Derivatives, etc.; Algebraic and Graphic Statements of the Conditions of Equilibrium with Applications; Scope of Pure Statics; Solutions of Statical Problems; Additional General Topics and Processes. Part II, Applications; Centers of Gravity; Stress; Structures; Stresses in Non-Framed Structures; Stresses in Framed Structures; Additional Topics and Examples; Appendices.

Gifts have also been received from the following:

Am. Ry. Assoc. 1 vol.
Am. Ry. Master Mechanics Assoc. 1 bound vol.
Am. Soc. Mech. Engrs. 1 bound vol., 1 vol.
Argollo, M. de T. e. 1 pam.
Belzner, Theodore. 1 vol.
Bezault, M. B. 3 pam.
Bissell, C. S. 1 pam.
Boston, Mass.-Eng. Dept. 3 pam.
Butler, M. J. 1 bound vol.
Cambridge, Mass.-City Engr. 6 pam.
Canada-Geol. Survey. 2 pam.
Canadian Min. Inst. 1 vol.
Chaumelin, Gaston. 1 vol.
Chemisches Laboratorium für Tonindustrie und Tonindustrie-Zeitung. 3 vol.
Columbia Univ. 1 vol.
Crane Co. 1 pam.
Decimal Assoc. 2 pam.
East Indian Ry. Co. 1 pam.
Farley, G. P. 1 bound vol.
Fetherston, J. T. 2 pam.
Fuller, W. B. 1 bound vol.
Illinois-Bureau of Labor Statistics. 1 bound vol.
Illinois-State Highway Comm. 1 bound vol.
Indian Midland Ry. Co. 1 pam.
Indiana-State Board of Health. 1 bound vol.
Inst. of Civil Engrs. 1 bound vol.
Inst. of Engrs. and Shipbuilders in Scotland. 1 bound vol., 1 pam.
Inter. Waterways Comm. 3 pam.
Iowa-Board of R. R. Commrs. 1 bound vol.
Krug, Edmundo. 3 pam.
Lakes-to-the-Gulf Deep Waterway Assoc. 1 pam.
Lehigh & Hudson River Ry. Co. 1 pam.
Lewis Inst. 1 pam.
Lochridge, E. E. 2 pam.
London, Ont.-Water Commrs. 1 pam.
Manchester Steam Users' Assoc. 1 pam.
Massachusetts-State Board of Health. 1 bound vol.
Mass. Inst. of Tech. 1 vol.
Master Car Builders' Assoc. 1 bound vol.
Merchants Assoc. of New York. 2 pam.
Mexican Central Ry. Co., Ltd. 1 pam.
Michigan-Secy. of State. 1 bound vol.
Montreal, Que.-City Clerk. 1 bound vol.
Morley, Fred. 1 bound vol., 1 pam.
Nat. Board of Fire Underwriters. 15 pam.
New York City-Board of Estimate and Apportionment. 1 pam.
New York City.-Public Service Comm. 1 bound vol., 2 pam.
New York City-Registrar of Records. 2 bound vol.
New York City Record. 1 bound vol.
New York, New Haven & Hartford R. R. Co. 2 pam.
Ohio-Bureau of Inspection and Supervision of Public Offices. 1 pam.
Ohio State Univ. 1 pam.

Oklahoma-Geol. Survey. 1 pam.
Pennsylvania State College. 1 pam.
Philadelphia, Pa.-Mayor. 3 bound vol.
Platt, T. C. 1 bound vol., 3 pam.
Poughkeepsie, N. Y.-Board of Public Works. 1 pam.
Powell, F. W. 1 pam.
Providence, R. I.-Dept. of Public Works. 10 pam.
Queensland, Australia-Commr. for Rys. 1 vol.
Richmond, Fredericksburg & Potomac R. R. Co. 1 pam.
Richardson, Clifford. 1 pam.
San Francisco, Cal.-City Engr. 1 vol.
Siemens-Schuckert Werke, G. m. b. H. 1 pam.
Smithsonian Institution. 1 bound vol., 1 pam.
Stanton, R. B. 2 diagrams.
Towne, H. R. 1 pam.
Tracy, Patrick J. 1 bound vol.
Union Pacific R. R. Co. 1 pam.
U. S.-Bureau of Insular Affairs. 1 pam.
U. S.-Bureau of the Census. 2 pam.
U. S.-Bureau of Steam Eng. 1 pam.
U. S.-Chief of Engrs. 12 specif.
U. S.-Coast and Geodetic Survey. 14 charts.
U. S.-Commr. of Education. 1 bound vol.
U. S.-Forest Service. 1 pam.
U. S.-Gen. Land Office. 1 bound vol.
U. S.-Geol. Survey. 1 pam.
U. S.-Isthmian Canal Comm. 1 vol.
U. S.-Library of Congress. 1 bound vol.
U. S.-Office of Experiment Stations. 5 bound vol.
U. S.-Office of Public Roads. 2 pam.
U. S.-Office of the Library and Naval War Records. 8 pam., 1 vol.
U. S.-Weather Bureau. 1 pam.
Univ. of Minnesota. 1 pam.
Victoria, Australia-Sludge Abatement Board. 6 pam.
Wallace, John F. 1 pam.
Washington Southern Ry. Co. 1 pam.
Western Australia-Govt. Rys. 1 pam.
Worcester Polytechnic Inst. 1 pam.

BY PURCHASE

Methods of Treating and Disposing of Sewage. Fifth Report, with Appendices 1, 2, 5, 6, 7, and 8 of the Royal Commission on Sewage Disposal. London, Wyman & Sons, Limited, 1908.

Illustrated Technical Dictionary in Six Languages. Edited by Alfred Schlomann. Vol. 4, Internal Combustion Engines; comp. by Carl Schikore. New York, McGraw Publishing Co., 1908.

Handbuch der Ingenieur Wissenschaften. Edited by Th. Landsberg. Part II, Der Brückenbau. Vol. 1 and 2, Fourth Edition, and Vol. 3 and 4, Third Edition. Part III, Der Wasserbau. Vol. 3, 5 and 8, Fourth Edition. Leipzig, Wilhelm Engelmann, 1904, '06–07.

Elektrotechnik in Einzeldarstellung. Edited by Gustav Benischke. Vol. 13, Elektrotechnische Messungen und Messinstrumente. By Gustav Wernicke. Braunschweig, Friedrich Vieweg und Sohn, 1909.

SUMMARY OF ACCESSIONS

From December 9th, 1908 to January 12th, 1909

Donations (including 18 duplicates)	201
By purchase	16
Total	217

MEMBERSHIP

ADDITIONS

(December 9th, 1908, to January 12th, 1909)

		Date of Membership.
MEMBERS		
BOOTH, GEORGE WILLIAM. Engr. in Chg. of Remodeling the Distribution System, N. Y. Dept. of Water Supply, 932 Park Row Bldg., New York City	Assoc. M. M.	Sept. 7, 1904 Nov. 2, 1908
BRYAN, FRED ASDEL. Gen. Mgr., Ind. & Mich. Elec. Co., 220 Colfax Ave., South Bend, Ind		Dec. 1, 1908
DOS SANTOS, JOSÉ AMERICO. Caixa 748, Rio de Janeiro, Brazil		Oct. 7, 1908
HIROI, ISAMI. Care, Mr. Miyoshi, 9 Thurleigh Rd., Wandsworth Common, London, S. W., England		Dec. 1, 1908
MCNAB, WILLIAM. Prin. Asst. Engr., Grand Trunk Ry. System, Montreal, Que., Canada		Dec. 1, 1908
WING, CHARLES BENJAMIN. Prof. of Structural Eng., Stanford Univ., 345 Lincoln Ave., Palo Alto, Cal	Assoc. M. M.	Nov. 4, 1896 Nov. 2, 1908
ASSOCIATE MEMBERS		
BLACK, EDWARD FRYLING. Instr. in Civ. Eng., Anglo-Chinese Coll., Foo Chow, China	Jun. Assoc. M.	Oct. 31, 1905 Nov. 4, 1908
BURRAGE, JOHN OTIS. Asst. Engr. to City Engr., San Francisco, Cal	Jun. Assoc. M.	Mar. 1, 1904 Nov. 4, 1908
COOKE, SAINT GEORGE HENRY. St. Geo. H. Cooke Co., Cambridge Bldg., Chester (Res., 608 Morton Ave., Ridley Park), Pa		Jan. 5, 1909
DARROW, MARIUS SCHOONMAKER. Chf. Engr., Irrigated Lands Co., 427 D. F. Walker Bldg., Salt Lake City, Utah	Jun. Assoc. M.	Feb. 6, 1900 Dec. 1, 1908
DIEHL, DAVID LESLIE. Engr. and Contr. (Whittaker & Diehl) and (Ferro Concrete Co.), Union Trust Bldg., Harrisburg, Pa		Jan. 5, 1909
ENSIGN, GUERT WILLIAM. Asst. State Highway Commr., Harrisburg, Pa		Nov. 4, 1908
GENTNER, OTTO HENRY, JR. Engr. in Chg. of Estimating and Designing, Gen. Fireproofing Co., Youngstown, Ohio		Nov. 4, 1908
KLEINSCHMIDT, HENRY SCHWING. Constr. Engr., Utah State Land Board, Salina, Utah	Jun. Assoc. M.	Mar. 1, 1904 Dec. 1, 1908
LANE, EDWARD PERCY. Asst. Engr., N. Y. C. & H. R. R. R., 131 West 72d St., New York City		Jan. 5, 1909
LARSON, CLARENCE MELROSE. 913 University Ave., Madison, Wis		Oct. 7, 1908

ASSOCIATE MEMBERS (*Continued*).

	Date of Membership.
McMORROW, JAMES WALTER. Secy., McMorrow Eng. & Const. Co., 360 West 125th St. (Res., 165 Audubon Ave.), New York City	Oct. 7, 1908
PERRY, FRANCIS WILLIAM. 117 Jerome St., Brooklyn, N. Y.	Jan. 5, 1909
ROUSSEAU, WILLIAM WHITE. Supt. of Constr., Bureau of Water Supply, 47 State St., Troy, N. Y.	Dec. 1, 1908
ROWLAND, WALTER. Draftsman, Isthmian Canal Comm., La Boca, Canal Zone, Panama	Dec. 1, 1908
SAMPLE, WILLIAM DWIGHT. Care, Madeira-Mamoré Ry., P. O. Box 304, Manaos, Brazil	Dec. 1, 1908
SKILLIN, EDWARD SIMEON. Second Vice-Pres., The Snare & Triest Co., 143 Liberty St., New York City	Dec. 1, 1908
TEIGEN, THOMAS WILLIAM ROSTAD. 192 West 5th St., St. Paul, Minn	Jun. Sept. 5, 1905 Assoc. M. Dec. 1, 1908
TURNER, AUGUSTUS MIESSE. 311 West 3d St., Greenville, Ohio	Dec. 1, 1908
WALTMAN, WILLIAM DEWITT. Supt., Porto Bello Rock Crushing Plants, Isthmian Canal Comm., Porto Bello, *via* Cristobal, Canal Zone, Panama	Dec. 1, 1908
WHEELER, ARTHUR CHAMBERS. U. S. Engr. Office, Honolulu, Hawaii	Oct. 7, 1908

ASSOCIATE

STROEBE, GEORGE GOTTLIEB. 315 New Eng. Bldg., Ann Arbor, Mich	Nov. 4, 1908

JUNIORS

ARMSTRONG, GEORGE SIMPSON, JR. 106 Penn St., Brooklyn, N. Y	Jan. 5, 1909
AYRES, LOUIS EVANS. With Gardner S. Williams, Cons. Engr., Ann Arbor, Mich	Dec. 1, 1908
CALHOUN, DAVID ADAMS. Chf. Engr., Richmond & Tide-Water Coal & R. R. Co., 29 Broadway, New York City	Oct. 6, 1908
CLEVELAND, LOU BAKER. Cleveland Bldg., Watertown, N. Y.	Sept. 1, 1908
EARL, AUSTIN WILLMOTT. 1050 Page St., San Francisco, Cal.	Dec. 1, 1908
FEIGEL, JOHN HENRY. Asst. Engr., Buffalo Southern Ry., Care, Frederick K. Wing, 910 White Bldg., Buffalo, N. Y	Oct. 6, 1908
LAKE, ORLOFF. 2032 State Ave., New Orleans, La	Dec. 1, 1908
McCRORY, THOMAS GEORGE. Asst. Engr., International Contract Co., Seattle, Wash	Dec. 1, 1908
MARTINEZ, ROLANDO ARNOLDO. Sn. Miguel 168, Havana, Cuba	Dec. 1, 1908
ROBERTSON, AVALON GRAVES. Asst. Engr., Changuinola Ry., United Fruit Co., Bocas del Toro, Panama	Oct. 6, 1908

RESIGNATIONS

MEMBERS	Date of Resignation.
MacRitchie, Charles	December 31, 1908
Whistler, Thomas Delano	December 31, 1908
ASSOCIATE MEMBERS	
Harper, Austin Curtin	December 31, 1908
Schenk, Carl Louis Eduard	December 31, 1908
ASSOCIATE	
Booth, William Ferris	December 31, 1908
JUNIORS	
Fernow, Rossiter Raymond	December 31, 1908
Jordan, Willis Robert	December 31, 1908
Snyder, William Henry	December 31, 1908

DEATHS

Carothers, Daniel Dawson. Elected Member, April 4th, 1894; died January 1st, 1909.

DuBarry, Edmund Louis. Elected Associate, January 6th, 1875; died December 4th, 1908.

Green, Rutger Bleecker. Elected Junior, May 5th, 1896; Associate Member, October 5th, 1898; Member, September 6th, 1904; died December 8th, 1908.

McCurdy, John Egbert. Elected Member, April 1st, 1896; died December 15th, 1908.

Prince, Edward. Elected Junior, February 6th, 1878; Member, November 1st, 1882; died December 12th, 1908.

MONTHLY LIST OF RECENT ENGINEERING ARTICLES OF INTEREST

(December 8th, 1908, to January 11th, 1909.)

NOTE.—*This list is published for the purpose of placing before the members of the Society, the titles of current engineering articles, which can be referred to in any available engineering library, or can be procured by addressing the publication directly, the address and price being given wherever possible.*

LIST OF PUBLICATIONS

In the subjoined list of articles, references are given by the number prefixed to each journal in this list:

(1) *Journal*, Assoc. Eng. Soc., 31 Milk St., Boston, Mass., 30c.
(2) *Proceedings*, Engrs. Club of Phila., 1317 Spruce St., Philadelphia, Pa.
(3) *Journal*, Franklin Inst., Philadelphia, Pa., 50c.
(4) *Journal*, Western Soc. of Engrs., Monadnock Blk., Chicago, Ill.
(5) *Transactions*, Can. Soc. C. E., Montreal, Que., Canada.
(6) *School of Mines Quarterly*, Columbia Univ., New York City, 50c.
(7) *Technology Quarterly*, Mass. Inst. Tech., Boston, Mass., 75c.
(8) *Stevens Institute Indicator*, Stevens Inst., Hoboken, N. J., 50c.
(9) *Engineering Magazine*, New York City, 25c.
(10) *Cassier's Magazine*, New York City, 25c.
(11) *Engineering* (London), W. H. Wiley, New York City, 25c.
(12) *The Engineer* (London), International News Co., New York City, 35c.
(13) *Engineering News*, New York City, 15c.
(14) *The Engineering Record*, New York City, 12c.
(15) *Railroad Age Gazette*, New York City, 15c.
(16) *Engineering and Mining Journal*, New York City, 15c.
(17) *Electric Railway Journal*, New York City, 10c.
(18) *Railway and Engineering Review*, Chicago, Ill., 10c.
(19) *Scientific American Supplement*, New York City, 10c.
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PROCEEDINGS

OF THE

AMERICAN SOCIETY

OF

CIVIL ENGINEERS

(INSTITUTED 1852)

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FEBRUARY, 1909.

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ON STEEL COLUMNS AND STRUTS: Austin L. Bowman, Alfred P. Boller, Emil Gerber, Charles F. Loweth, Ralph Modjeski, Frank C. Osborn, George H. Pegram, Lewis D. Rights, George F. Swain, Emil Swensson, Joseph R. Worcester.

The House of the Society is open from 9 A.M. to 10 P.M. every day, except Sundays, Fourth of July, Thanksgiving Day and Christmas Day.

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AMERICAN SOCIETY OF CIVIL ENGINEERS

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PROCEEDINGS

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MINUTES OF MEETINGS

OF THE SOCIETY

FIFTY-SIXTH ANNUAL MEETING.*

January 20th, 1909.—The meeting was called to order at 10 A. M.; President Charles Macdonald in the chair; Chas. Warren Hunt, Secretary; and present, also, about 250 members.

The reading of the minutes of January 6th, 1909, was dispensed with.

Messrs. George W. Parsons, Shirley C. Hulse, and Herbert E. Cantwell were appointed tellers to canvass the Ballot for Officers for the ensuing year.

The Annual Report of the Board of Direction, and the Annual Reports of the Secretary and of the Treasurer,† for the year ending December 31st, 1908, were presented and accepted.

* A full report of the Fifty-sixth Annual Meeting is printed on pages 54 to 70 of this number of *Proceedings*.

† For these reports, see pages 9 to 17 of *Proceedings* for January, 1909 (Vol. XXXV).

The Secretary reported that, in accordance with the unanimous report of the Committee to Recommend the Award of Prizes, the Board of Direction had awarded the prizes for the year ending with the month of July, 1908, as follows:

The Norman Medal to Charles C. Schneider, Past-President, Am. Soc. C. E., for his paper entitled "Movable Bridges."

The Thomas Fitch Rowland Prize to Edward E. Wall, M. Am. Soc. C. E., for his paper entitled "Water Purification at St. Louis, Mo."

The Collingwood Prize for Juniors to D. W. Krellwitz, Jun. Am. Soc. C. E., for his paper entitled "Reinforced Concrete Towers."

The Committee also reported in favor of taking some official notice of the paper entitled "Effects of the San Francisco Earthquake of April 18th, 1906, on Engineering Constructions," prepared by sub-committees of the San Francisco Association of Members of the American Society of Civil Engineers, and the Secretary was directed to express to the members of that Association the appreciation of the Board and of the Society of the interesting and valuable papers presented.

The Secretary announced the decision of the Board of Direction to hold the next Annual Convention at the Mount Washington Hotel, Bretton Woods, N. H., from July 6th to 9th, 1909, inclusive.

The Secretary announced the appointment of the following members on the Special Committee to consider and report upon "The Design, Ultimate Strength, and Safe Working Values of Steel Columns and Struts": A. L. Bowman, Lewis D. Rights, Alfred P. Boller, Emil Gerber, Charles F. Loweth, Ralph Modjeski, Frank C. Osborn, George H. Pegram, George F. Swain, Emil Swensson, and Joseph R. Worcester.

A. L. Bowman, Chairman of the Special Committee on Columns and Struts, submitted a preliminary report,* containing a resolution to the effect that the United States Government be requested to make a sufficient appropriation for and proceed with the construction of a testing machine for full-sized bridge members, and that the Secretary be directed to forward copies of the resolution to the President of the United States, the Vice-President, and the Speaker of the House of Representatives.

The report was accepted and the resolution adopted.

The following were appointed members of the Nominating Committee to serve two years:

Frederick W. Gardiner. .	*Representing*	*District*	*No.*	*1.*
Richard A. Hale.....	"	"	"	*2.*
Edward B. Codwise.....	"	"	"	*3.*
Frank Sutton.....	"	"	"	*4.*
H. E. Riggs.....	"	"	"	*5.*
Charles C. Wentworth.	"	"	"	*6.*
C. E. Fowler.....	"	"	"	*7.*

* See page 56.

The Secretary presented a report from the Special Committee on the Metric System.*

The report was accepted and ordered printed, and the Committee was discharged with the thanks of the Society.

The Secretary presented Majority and Minority reports from the Special Committee on Concrete and Reinforced Concrete.†

It was ordered that the Majority and Minority Reports be printed for distribution to the membership, that they be made an order of business for discussion at the next Annual Convention, and that the Committee be continued.

The Secretary presented a progress report‡ from the Special Committee on Uniform Tests of Cement.

The report was accepted and the Committee continued.

The Secretary presented a progress report§ from the Special Committee on Rail Sections.

The report was accepted and the Committee continued.

The Secretary presented a letter‖ from the Chairman of the Special Committee on Engineering Education.

The Secretary presented a report¶ from the Committee appointed at the Denver Convention to consider and report upon certain amendments to the Constitution, proposed by Mr. Clemens Herschel and others, which recommended that the amendment be "re-worded by the Secretary to apply to the Constitution as it now exists, and sent out to letter-ballot with an accompanying statement that it must be voted upon by letter-ballot to meet the requirements of the Constitution; but that in view of the adoption of an amendment since its presentation placing the election of members in the hands of the Board of Direction, it is the opinion of this Committee that it should be defeated."

The report was accepted and its recommendations adopted.

The Secretary made some announcements in reference to the programme of the Annual Meeting.

Samuel Whinery, M. Am. Soc. C. E., presented the following preamble and resolution:

"*Whereas:* The present method of appointing the Nominating Committee of the Society has, in practice, proved to be unrepresentative and unsatisfactory, and

"*Whereas:* Section 2 of Article VII of the Constitution provides that the Board of Direction may prescribe the mode of procedure for appointing the Nominating Committee, therefore be it

* This report was printed in *Proceedings*, Vol. XXXIV, page 415 (October, 1908).
† See page 85.
‡ See page 71.
§ See page 61.
‖ See page 63.
¶ See page 64.

"*Resolved:* That the Board of Direction be requested to give further consideration to the whole subject of the method of nominating officers for the Society; and particularly to the present mode of procedure in ascertaining and recording an expression of the corporate membership with regard to the choice of members of the Nominating Committee in the several geographical districts; and to report its conclusions and recommendations to the next Annual Convention of the Society."

The resolution was adopted.

The report of the tellers* appointed to canvass the Ballots for Officers for the ensuing year was presented.

The President announced the election of the following officers:

President, to serve one year:

ONWARD BATES, Chicago, Ill.

Vice-Presidents, to serve two years:

GEORGE HERNDON PEGRAM, New York City.
EMIL SWENSSON, Pittsburg, Pa.

Treasurer, to serve one year:

JOSEPH M. KNAP, New York City.

Directors, to serve three years:

FRANCIS LEE STUART, New York City.
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ARTHUR NEWELL TALBOT, Urbana, Ill.
WILLIAM MONTGOMERY GARDNER, Memphis, Tenn.
HORACE AUGUSTUS SUMNER, Denver, Colo.

Mr. Benzenberg and Mr. Stearns conducted Mr. Bates, the President-elect, to the chair.

Mr. Bates addressed the meeting briefly.

Adjourned.

February 3d, 1909.—The meeting was called to order at 8.30 P. M.; James Owen, M. Am. Soc. C. E., in the chair; Chas. Warren Hunt, Secretary; and present, also, 108 members and 12 guests.

The minutes of the meetings of December 16th, 1908, and January 6th, 1909, were approved as printed in the *Proceedings* for January, 1909.

A paper by Robert Spurr Weston, Assoc. M. Am. Soc. C. E., entitled "The Purification of Ground-Waters Containing Iron and Manganese", was presented by the author and illustrated with lantern slides and chemical experiments. In making the experiments the

*See page 69.

author was assisted by Mr. D. D. Jackson, Chemist for the New York City Department of Water Supply, Gas and Electricity.

The paper was discussed by Emil Kuichling, M. Am. Soc. C. E., and the author, and written communications from Messrs. W. C. Lounsbury, and Arthur L. Terry, Jr., were read by the Secretary.

The Secretary announced the election of the following candidates by the Board of Direction on February 2d, 1909:

As Members.

Earl Ivan Brown, Wilmington, N. C.
George Mairs Bull, Denver, Colo.
John Angell Fox, Cincinnati, Ohio.
Walter Hayden Graves, Spokane, Wash.
James Charles Haugh, New Orleans, La.
George Pierce Howell, Manila, Philippine Islands.
Richard Warren Knight, Pittsburg, Pa.
William Walter Marr, Chicago, Ill.
John William Moore, Rushville, Ind.
Daniel William Murphy, Klamath Falls, Ore.
George Nelson Randle, Sacramento, Cal.
Orville Hickman Turner, Raton, N. Mex.
Meriwether Lewis Walker, Manila, Philippine Islands.

As Associate Members.

Albert Cornelius Arend, Omaha, Nebr.
McCrea Parker Blair, Saint Boniface, Man., Canada.
Robert Lambert Fowler, Perth Amboy, N. J.
Kenneth Crothers Grant, Harrisburg, Pa.
James Robert Hardesty, Pottsville, Pa.
Lauren Augustus Pettebone, Johnsonville, N. Y.
Edmund Phipps Sanger, Mt. Vernon, N. Y.
Leroy Tallman, Portsmouth, R. I.
Granville Lewis Taylor, Wilkinsburg, Pa.
Walter Robert Wheaton, Washington, D. C.
Charles Newton Young, San Francisco, Cal.

As Associate.

Albert Henry Bromley, Jr., Philadelphia, Pa.

As Juniors.

Josiah Richardson Brooks, Pt. Pleasant, W. Va.
Irvin Sutton Grindrod, Philadelphia, Pa.
Frank Alden Russell, Peabody, Kans.
Robert John Schmid, Spokane, Wash.
Charles Randolph Simpson, Warren, Pa.
Joseph Smith, New York City.
William Rogers Tyler, New York City.

The Secretary announced the transfer of the following candidates by the Board of Direction on February 2d, 1909:

FROM ASSOCIATE MEMBER TO MEMBER.

HERBERT CLARENDON ALDEN, New York City.
ROMEO THOMPSON BETTS, New York City.
FERDINAND FINNEY HARRINGTON, Norfolk, Va.
GAVIN NELSON HOUSTON, Denver, Colo.
ELMER GOVE MANAHAN, New York City.
RALPH EELLS NEWTON, Milwaukee, Wis.
FREDERICK CELESTINE SCHUBERT, Portland, Ore.

FROM JUNIOR TO MEMBER.

ERLE LEROY VEUVE, Los Angeles, Cal.

FROM JUNIOR TO ASSOCIATE MEMBER.

ARTHUR MERRICK NEWBERRY BLAMPHIN, New Orleans, La.
WALTER GOTTLIEB FEDERLEIN, Mogollon, N. Mex.
DIEDRICH WILLIAM KRELLWITZ, West New York, N. J.
FULTON PACE, Guayama, Porto Rico.
JASON PAIGE, Chicago, Ill.
ALBERT ORANGE SMITH, Port Jefferson, N. Y.

FROM JUNIOR TO ASSOCIATE.

GEORGE MERRICK HERRON, Palo Alto, Cal.

The Secretary announced the election of the following candidates by the Board of Direction on September 1st, 1908:

AS JUNIORS.

JOHN CHARLES PRICHARD, Columbia, Mo.
HAROLD S. WILLIAMS, Caldwell, Idaho.

The Secretary announced the following deaths:

GEORGE ADGATE, elected Member April 1st, 1896; died January 25th, 1909.

WILLIAM CREELMAN AMBROSE, elected Member, April 4th, 1888; died January 3d, 1909.

DANIEL DAWSON CAROTHERS, elected Member, April 4th, 1894; died January 7th, 1909.

WILLIAM PRICE CRAIGHILL, elected Member, October 7th, 1885; Honorary Member, March 23d, 1896; Director of the Society during the years 1892-93; President, 1894; died January 18th, 1909.

Baron EMILE THEODORE QUINETTE DE ROCHEMONT, elected Member, January 3d, 1894; died December 8th, 1908.

Adjourned.

OF THE BOARD OF DIRECTION

(Abstract)

January 20th, 1909.—The Board met, as required by the Constitution, at the House of the Society during the Annual Meeting, January 20th, 1909, at 12.30 P. M., President Bates in the chair, Chas. Warren Hunt, Secretary, and present, also, Messrs. Andrews, Benzenberg, Brackett, Christie, Churchill, Harrison, Hazen, Hodgdon, Knap, Macdonald, Pegram, Schneider, Stearns, Swensson, Talbot, Tillson, Wilkins, and Williams.

The President announced the first business of the Board to be the election of a Secretary.

Mr. Hunt retired.

Chas. Warren Hunt was nominated for Secretary, and every member present voted for his election. The President declared Mr. Hunt unanimously elected Secretary.

Mr. Hunt returned and resumed his duties.

The following Standing Committees of the Board were appointed:

Finance Committee: George H. Pegram, S. C. Thompson, Allen Hazen, W. G. Wilkins, M. T. Endicott.

Publication Committee: Charles L. Harrison, Francis Lee Stuart, George F. Swain, Emil Swensson, Horace Andrews.

Library Committee: George W. Kittredge, James Christie, F. W. Hodgdon, Dexter Brackett, Chas. Warren Hunt.

A Committee on Membership was also appointed.

Adjourned.

February 2d, 1909.—Vice-President Pegram in the chair; Chas. Warren Hunt, Secretary, and present also Messrs. Andrews, Hodgdon, Kittredge, Knap, Noble, Schneider, Stuart, Thompson, and Tillson.

Ballots for Membership were canvassed, resulting in the election of 13 Members, 11 Associate Members, 1 Associate, and 7 Juniors, the transfer of 7 Associate Members to the grade of Member, 1 Junior to the grade of Member, 6 Juniors to the grade of Associate Member, and 1 Junior to the grade of Associate.

The following Committees were appointed to take charge of the next Annual Convention:

Committee of the Board of Direction: Messrs. Hodgdon, Tillson, and Hunt.

Local Committee: Messrs. H. W. Hayes, A. W. Dean, S. E. Tinkham, H. D. Woods, J. F. Stevens, Geo. A. Kimball, and J. W. Ellis.

The resolution adopted by the Annual Meeting and relating to the method of the appointment of the Nominating Committee was considered.

Applications were considered, and other routine business transacted.

Adjourned.

REPORT IN FULL OF THE FIFTY-SIXTH ANNUAL MEETING, JANUARY 20th and 21st, 1909.

Wednesday, January 20th, 1909.—President Macdonald in the Chair; Chas. Warren Hunt, Secretary; and present, also, about 250 members.

Meeting called to Order.

THE PRESIDENT.—The meeting will come to order. According to the Constitution, the ballot for officers will close at twelve o'clock noon. The polls will be left open until that time. The tellers, in the meantime, will proceed with the canvass, in order to be able to make a report before the close of the meeting. I will announce the tellers later, and we will hear the report of the Board of Direction.

Report of the Board of Direction.

The Secretary read the Report of the Board of Direction.*

Tellers appointed.

THE PRESIDENT.—The following gentlemen are requested to serve as tellers, H. E. Cantwell, S. C. Hulse, and George W. Parsons.

THE SECRETARY.—If any of these gentlemen are present, I wish to say that arrangements have been made for the canvass of the ballots on the third floor, and a number of assistants in the office will aid in opening the ballots (of which there are about 1 100) so that they can get through quickly.

Shall I read the report of the Secretary, sir?

THE PRESIDENT.—Yes.

Report of the Secretary.

The Secretary read his Report of Receipts and Disbursements.†

THE SECRETARY.—This report shows that notwithstanding the fact that we have paid $10 000 on the mortgage there is a balance of some $12 000 or $13 000 more than a year ago. The balance sheet, Mr. President, which accompanies this report, and is in the hands of all members, seems to indicate that the available assets of the Society are somewhat over $300 000, about $370 000, if the value of the books in the library is included.

THE PRESIDENT.—Have you the Treasurer's Report?

Report of the Treasurer.

The Secretary read the Report of the Treasurer.‡

THE PRESIDENT.—If there is no objection, the reports read by the Secretary will be accepted and passed to file.

THE SECRETARY.—The following report was received by the Board of Direction from the Committee to Recommend the Award of Prizes.

Award of Prizes.

"THE BOARD OF DIRECTION,
"American Society of Civil Engineers,
"220 West 57th Street, New York City, N. Y.

"GENTLEMEN:

"The undersigned, appointed by letter of the Secretary of the Society dated June 15, 1908, members of a committee to recommend the award of prizes for papers presented to the Society for the year

*See *Proceedings*, Vol. XXXV, p. 9 (January, 1909).
†See *Proceedings*, Vol. XXXV, p 14 (January, 1909).
‡See *Proceedings*, Vol. XXXV, p. 17 (January, 1909).

ending with the month of July, 1908, have the honor to report that having given the matter due consideration we unanimously recommend

"That the Collingwood prize for Juniors be awarded to Article No. 1066, 'Reinforced Concrete Towers' by D. W. Krellwitz;

"That the Rowland Prize be awarded to Paper No. 1067, 'Water Purification at St. Louis, Mo.' by Edward E. Wall;

That the Norman Medal be awarded to No. 1071, 'Movable Bridges' by C. C. Schneider.

"Paper No. 1056, 'Effect of the San Francisco Earthquake of April 18th, 1906, on Engineering Constructions' contains articles of great interest and merit prepared by sub-committees of the San Francisco Association of Members of the American Society of Civil Engineers, and we are of opinion that it would be appropriate and graceful for the Society to take official notice of this paper. Although this matter lies outside the duty assigned to us, we invite the attention of the Board of Direction to it, for such action as the Board may think appropriate.

"JOHN W. ALVORD (Chicago, Ill.),
"C. H. MCKINSTRY (San Francisco, Cal.),
"GEORGE G. ANDERSON (Denver, Colo.)."

THE SECRETARY.—The Board of Direction has awarded the prizes to the gentlemen named, as recommended by the Committee, and has also, in accordance with the recommendation of the Committee, expressed to the members of the San Francisco Association, the appreciation of the Board and of the Society of the interesting and valuable papers presented.

THE PRESIDENT.—The report as to the time and place of the next Convention, Mr. Secretary?

Time and Place of Annual Convention.

THE SECRETARY.—The Board of Direction has decided, Mr. President, that the next Annual Convention of the Society shall be held at the Mount Washington Hotel, Bretton Woods, N. H., from July 6th to 9th, 1909, inclusive.

I might say that that hotel will accommodate at least 500, and we are to have the exclusive use of it. In case there should be 600 or 700 there is another hotel, under the same management, near by, which can take care of the overflow, and if the attendance should run up to 800 or 900, the Fabian House is only about half a mile away, so there appears to be no doubt that all the members who want to go to this Convention and their families can be accommodated.

Special Committee on Columns and Struts.

I have to announce, Mr. President, that the Special Committee authorized by vote of the Society to consider and report upon "The Design, Ultimate Strength and Safe Working Values of Steel Columns and Struts" has been appointed by the Board and has organized as follows: A. L. Bowman, Chairman, Lewis D. Rights, Secretary, together with the following members making up the committee, Alfred P. Boller, Emil Gerber, Charles F. Loweth, Ralph Modjeski, Frank C.

Special Committee on Columns and Struts (continued).

Osborn, George H. Pegram, George F. Swain, Emil Swensson, and Joseph R. Worcester.

A. L. BOWMAN, M. AM. SOC. C. E.—Mr. President, the Special Committee appointed to investigate and report upon "The Design, Ultimate Strength and Safe Working Values of Steel Columns and Struts" reports as follows to the Members of the American Society of Civil Engineers in Annual Meeting:

Report of Special Committee on Columns and Struts.

TO THE MEMBERS OF THE AM. SOC. C. E.
IN ANNUAL MEETING:

The Special Committee authorized by vote of this Society "To consider and report upon the design, ultimate strength, and safe working values of steel columns and struts," having recently organized, desires to emphasize the national importance and necessity of compression tests, and recommends that the following resolution be adopted by this meeting:

WHEREAS, no full size tests on large compression members have been made, there being no testing machine of sufficient magnitude for the purpose, and

WHEREAS, the necessity for such tests has been fully established, and the results obtained from them would add greatly to engineering knowledge and be of material benefit to the industries of this country, and

WHEREAS, it is the sense of the American Society of Civil Engineers in Annual Meeting assembled, that the building of a machine capable of testing to destruction full size compression members of large dimensions, and of accurately recording results, is beyond the means of private interests, and can best be undertaken by the United States Government.

RESOLVED, that the United States Government be hereby requested to make a sufficient appropriation for and proceed with the construction of a testing machine which will accomplish the desired results.

RESOLVED, that the Secretary be directed to forward copies of this resolution to the President of the United States, the Vice-President and the Speaker of the House of Representatives.

For the Committee,

A. L. BOWMAN,
Chairman.

CHARLES B. BALL, M. AM. SOC. C. E.—This would seem to be a matter which the Federal Government only can handle properly, and I am sure that such a mild recommendation as this Committee proposes ought to be generally acceptable to this meeting. I therefore move that the report of the Committee—I presume it is merely a preliminary report—be accepted, and that the resolution as recommended be passed.

The motion was duly seconded.

THE PRESIDENT.—Gentlemen, you have heard the motion. It has been duly seconded. Is there any discussion upon the subject? All

in favor of that motion will signify it by saying "aye"; contrary, "no." It is a vote.

THE SECRETARY.—Mr. President, the next business is the appointment by this meeting of seven members of the Society as members of the Nominating Committee, one from each of the seven geographical districts into which the Society is divided for the purpose of the Nominating Committee. Nominating Committee.

In District No. 1 the total number of suggestions received is 233. Mr. Frederick W. Gardiner has received 148 votes; Mr. George S. Rice, 20; *Mr. J. G. Gardiner, 4; Mr. Arthur S. Tuttle, 3; Mr. Henry W. Hodge, 3; and Messrs. James H. Edwards, Nelson P. Lewis, Theodor S. Oxholm and George C. Whipple have received 2 votes each.

The following have received one vote each:

W. H. BURR,
L. BUSH,
C. E. CARPENTER,
A. B. CORTHELL,
ALBERT S. CRANE,
RICHARD T. DANA,
H. F. DUNHAM,
LORING N. FARNUM,
GEORGE B. FRANCIS,
ALBERT I. FRYE,
E. P. GOODRICH,
CHARLES S. GOWEN,
CHARLES H. GRAHAM,
L. E. GREGORY,
C. L. HARRISON,
G. A. HARWOOD,
W. J. HASKINS,
A. L. A. HIMMELWRIGHT,
ALLEN HAZEN,
CARY T. HUTCHINSON,
C. M. INGERSOLL, JR.,
GORGE A. JUST,
*LEONARD METCALF,
EDWARD P. NORTH,
H. L. OESTREICH,
C. J. PARKER,
J. R. SAVAGE,
M. R. SHERRERD,
J. WALDO SMITH,
JOSEPH STRACHAN,
EDWIN THACHER,
JOHN G. VAN HORNE,
JOHN C. WAIT,
*D. A. WATT,
CHARLES E. WELLS,
SAMUEL WHINERY,
W. J. WILGUS.

It was duly moved and seconded that Mr. Frederick W. Gardiner be elected a member of the Nominating Committee from District No. 1.

THE PRESIDENT.—It is moved and seconded that Mr. Gardiner be made a member of the Nominating Committee from District No. 1. All in favor of that signify by saying "aye"; contrary, "no." It is a vote.

THE SECRETARY.—From District No. 2 the total number of votes received is 54. Mr. Richard A. Hale has received 13 votes; Mr. J. R. Worcester, 12; Mr. Harrison P. Eddy, 5; and Messrs. Charles A. Allen,

*Of the above, J. G. Gardiner is not a member of the Society, and Messrs. Leonard Metcalf and D. A. Watt do not reside in District No. 1.

Nominating Committee (continued).

Desmond FitzGerald, Hector J. Hughes, Hiram A. Miller, Albert L. Scott and *George F. Swain have received 2 votes each.

The following have received one vote each:

H. K. Barrows,	George A. Kimball,
George A. Carpenter,	Henry Manley,
Fayette S. Curtis,	Leonard Metcalf,
J. R. Freeman,	William H. Moore,
Edward Gagel,	O. Perry Sarle,
*C. M. Ingersoll, Jr.,	*F. P. Stearns.

Frederic P. Stearns, Past-President, Am. Soc. C. E.—I move that Mr. Richard A. Hale be elected a member of the Nominating Committee from District No. 2.

The motion was duly seconded.

The President.—You have heard the motion, gentlemen. All in favor signify by saying "aye"; contrary, "no." It is a vote.

Joseph M. Knap, Treasurer, Am. Soc. C. E.—I move that hereafter only the first three names be read. It is not worth while perhaps to read the names of those who get only one vote, and it would expedite things if only the first three names were read.

The President.—If there is no objection that seems to be a very good suggestion for the Secretary to observe.

The Secretary.—From District No. 3 the total number of votes received was 111. Mr. M. G. Barnes has 30 votes; Mr. Edward B. Codwise, 27; and Mr. E. E. Haskell, 7.

Carlton E. Davis, M. Am. Soc. C. E.—Mr. President, I move that Mr. Codwise be appointed a member of the Nominating Committee; though he has not received as many votes as Mr. Barnes, yet I think that his appointment will serve better to represent the District. There is one member of the Nominating Committee from District No. 3 who has still a year to serve, Mr. David A. Watt, who is a member of the State Engineer's Office at Albany. Mr. Barnes is also a member of the State Engineer's Office at Albany, so that it seems to me that the election of Mr. Codwise will be of more universal service to the Society, and not concentrate the entire representation in the State Engineer's Office, at Albany.

The motion was duly seconded.

Joseph Ripley, M. Am. Soc. C. E.—I move to amend that motion by substituting the name of M. G. Barnes.

The motion was duly seconded.

The President.—The vote on the amendment will come first in the natural course.

The Secretary.—Would it not be simpler, Mr. President, to withdraw all motions and to vote on each of the names.

*Of the above, Messrs. George F. Swain and F. P. Stearns are not eligible, being members of the Board of Direction; and C. M. Ingersoll, Jr., does not reside in District No. 2.

MR. WEBSTER.—I make a motion to that effect.

THE PRESIDENT.—You have heard the motion that we take a vote on the two names mentioned from the Third District. All in favor of that signify by saying "aye"; contrary, "no." It is a vote.

MR. KNAP.—I move that it be a rising vote.

THE PRESIDENT.—All in favor of Mr. Barnes's election will please rise.

A MEMBER.—Will you kindly state what the motion is?

THE PRESIDENT.—Mr. Barnes's name will now be voted upon. All in favor of his appointment from the Third District will please rise.

THE SECRETARY.—Twenty.

THE PRESIDENT.—All in favor of Mr. Codwise will please rise.

THE SECRETARY.—Sixty-seven.

THE PRESIDENT.—I declare Mr. Codwise duly elected from District No. 3.

THE SECRETARY.—From District No. 4 the total number of votes received is 110. Mr. Frank Sutton has 40 votes; Mr. Richard L. Humphrey, 19; Mr. Calvin W. Hendrick, 4.

MR. DAVISON.—I move that Mr. Frank Sutton be elected as member from District No. 4.

The motion was duly seconded.

THE PRESIDENT.—You have heard the motion. All in favor signify by saying "aye"; contrary, "no." It is a vote.

THE SECRETARY.—From District No. 5 there are 126 votes, of which Mr. H. E. Riggs has 58; Mr. O. W. Childs, 25; and Mr. Archibald S. Baldwin, 3.

It was duly moved and seconded that Mr. Riggs be appointed a member of the Nominating Committee from District No. 5.

THE PRESIDENT.—All in favor of that say "aye"; contrary, "no." It is a vote.

THE SECRETARY.—From District No. 6 the total number of votes received is 79, of which Mr. Charles C. Wentworth has received 9; Mr. J. F. Coleman, 5; Mr. Hunter McDonald, 4. In this case I have to read the fourth name because Mr. Frank M. Kerr has also received 4.

C. S. CHURCHILL, M. AM. SOC. C. E.—I nominate Mr. Charles C. Wentworth to be a member of the Nominating Committee from District No. 6.

The motion was duly seconded.

THE PRESIDENT.—It is moved and seconded that Mr. Wentworth be elected a member of the Nominating Committee from District No. 6. All in favor will say "aye"; contrary, "no." It is a vote.

THE SECRETARY.—From District No. 7 the total vote is 108; of which Mr. C. E. Fowler has received 10; Mr. H. J. Burt, 7; Mr. Edwin Duryea, Jr., 7; and Mr. R. H. Thomson, 7.

GEORGE H. PEGRAM, M. AM. SOC. C. E.—I move that Mr. Fowler

Nominating Committee (continued).

be elected as a member of the Nominating Committee from District No. 7.

The motion was duly seconded.

THE PRESIDENT.—It is moved and seconded that Mr. Fowler be elected a member of the Nominating Committee from District No. 7. All in favor of that signify by saying "aye"; contrary "no." It is a vote.

Report of Special Committee on Status of the Metric System.

THE SECRETARY.—The next order of business is the report of the Committee on the Status of the Metric System. This report was published in the *Proceedings* for October, 1908, and copies, in pamphlet form, are available for distribution to those in attendance at this meeting.

I also have a discussion and criticism of this report from Mr. Henry R. Towne, who is not a member of the Society; but if I might venture to make a suggestion, this Committee has made what is intended to be a final report, and wishes to be discharged. Now, whether this report should be printed without any discussion or not is a matter for this meeting to determine, but it does seem to me that this report might be treated as a paper of the Society, and published with the discussions to which it gives rise, because there is a good deal of difference of opinion as to the availability of the metric system, and it might lead to something.

THE PRESIDENT.—The correct course, gentlemen, would be to move to discharge the committee with thanks, and print the report.

It was moved and seconded that the Committee be discharged with thanks, and that the report be printed.

THE PRESIDENT.—You have heard the motion, duly seconded, that this Committee be discharged, with the thanks of the Society, and that its report be printed. All in favor of that signify by saying "aye"; contrary, "no." It is a vote.

Report of Special Committee on Concrete and Reinforced Concrete.

THE SECRETARY.—The next business in order is the Report of the Special Committee on Concrete and Reinforced Concrete.* This report, which is signed by six members of the Committee, and a minority report signed by the other four members, is available in pamphlet form for distribution to those in attendance, and perhaps need not be read. Mr. Humphrey, the Secretary of the Committee, is here, I believe.

RICHARD L. HUMPHREY, M. AM. SOC. C. E.—Mr. President, the report cited is the report agreed upon by a majority of the Special Committee on Concrete and Reinforced Concrete, and is also the report agreed upon by the Joint Committee appointed by the American Society for Testing Materials, the American Railway Engineering and Maintenance of Way Association, and the Association of American

*The majority and minority reports of this Committee are printed on pages 85 to 120 of this number of *Proceedings*.

Portland Cement Manufacturers, with which your Special Committee co-operated, and it is printed for the purpose of getting discussion on it. The Committee is not able to present a final report at this time.

The report itself is presented for the reason that the majority of the members of the committee believe that the information which the committee has acquired and the views expressed would be of value and interest to the members of the Society. The committee desires that the report be printed and circulated among the members and that the committee be continued.

A MEMBER.—I second that motion.

THE SECRETARY.—I call attention to the fact that there is a Minority Report, which is not very long, and ask whether it is the pleasure of the meeting that it be read.

THE PRESIDENT.—I understand that the gentleman's motion is that the Majority and Minority reports be printed and distributed among the members, leading up to a discussion at some future meeting, and that the Committee be continued.

THE SECRETARY.—Might I venture to suggest, Mr. President, that it might be wise to insert in that motion the date on which the discussion should be taken up, and also that the next Annual Convention would be a very good time.

A MEMBER.—I accept that.

The motion was duly seconded.

THE PRESIDENT.—You have heard that motion. All in favor say "aye"; contrary, "no." The ayes appear to have it; the ayes have it. It is a vote.

Report of Special Committee on Uniform Tests of Cement.

THE SECRETARY.—The Report of the Special Committee on Uniform Tests of Cement is here in pamphlet form, and I will simply read the last few lines of it.*

A MEMBER.—I move that the Report be received and the Committee continued.

Motion duly seconded.

THE PRESIDENT.—You have heard the motion. All in favor of it signify by saying "aye"; contrary, "no." It is a vote.

Report of Special Committee on Rail Sections.

THE SECRETARY.—I have the following brief report from the Special Committee on Rail Sections:

DECEMBER 18TH, 1908.

AMERICAN SOCIETY OF CIVIL ENGINEERS,

GENTLEMEN:—Your Committee respectfully reports that since its report of December 6th, 1907, it has continued its investigations, keeping in touch with the progress made by railroads, and the other engineering associations of the country, as well as experiments and tests made by the United States Government at the Watertown Arsenal, Watertown, Mass.

The progress made on this subject has been slow, but quite im-

* This report is printed on pages 71 to 84 of this number of *Proceedings*.

Report of Special Committee on Rail Sections (continued).

portant. The information gathered is quite voluminous—so much so that it is hardly necessary to burden this Society with much of it, as there is no body of men, in railroad circles or in the different societies, which is ready to-day to make a recommendation for a rail section to be adopted by all the railroads. Your Committee is therefore not yet prepared to submit respective sections for the different weights of rail. The cardinal principles submitted in your Committee's Report of December 6th, 1907, however, seem to have been generally accepted by all professional men and railroads making a study of the subject, and so far as sctions of rail are being made we believe they are accepted as good and being adhered to, and it is gratifying to your Committee to be able so to report.

We may add that since our last Report the American Railway Association has placed its work in the hands of a Committee of Operating Engineers of the American Railway Engineering and Maintenance of Way Association of Chicago, and asked that Committee to report on a rail section, this report to be returned to the American Railway Association to be considered for adoption. Three members of your Committee are members of the Committee of the American Railway Engineering and Maintenance of Way Association, and a greater number of them are members of the American Railway Association. Also, the American Society for Testing Materials has a Committee, several of the members of which are members of your Committee, so that the work of the several Committees is interwoven to such an extent that they are working in harmony.

The American Society for Testing Materials, since our report, has made some important changes in its specifications, bringing them more into harmony with the specifications proposed by this Committee.

We also desire to call attention to the increased use of open-hearth steel rails, and of the greater facilities we will have for obtaining open-hearth steel rails in the future,

These Associations have also joined in collecting information as to rail failures, and have adopted a uniform blank for reporting rail failures, which will give information much more in detail and more accurate than the information that has generally been collected by the railroads hitherto.

They have also joined in having tests made in ingots, blooms, rails and rail joints, under the supervision of the United States Arsenal at Watertown, Mass. All this work will add very much to the data available in the study of the making of rails at the mills, and the wear and failure of them on the railroads; and, considering the situation as it stands, your Committee would deem it wise to make this a Report of Progress, and ask to be continued.

We respectfully remain,

JOSEPH T. RICHARDS, *Chairman.*
ROBERT W. HUNT, *Secretary.*
C. W. BUCHHOLZ,
E. C. CARTER,
S. M. FELTON,
JOHN D. ISAACS,
RICHARD MONTFORT,
H. G. PROUT,
PERCIVAL ROBERTS, JR.,
GEORGE E. THACKRAY,
EDMUND K. TURNER,
WILLIAM R. WEBSTER.

THE PRESIDENT.—What is your pleasure in regard to this report?

It was duly moved and seconded that the Report be received and the Committee continued.

THE PRESIDENT.—You have heard the motion that the Report be received and the Committee continued. All in favor signify by saying "aye"; contrary, "no." It is a vote.

Special Committee on Engineering Education.

THE SECRETARY.—Mr. President, from the Chairman of the Special Committee on Engineering Education, Mr. Desmond FitzGerald, I have received the following note:

"BROOKLINE, MASS.,
16 JANUARY, 1909.

"MY DEAR MR. HUNT

"As a partial progress report of the 'Education Committee' I send you these few lines. During last year a Joint Committee on Engineering Education has been formed consisting of the following members:

"Representing Com. of Am. Soc. C. E.	Desmond FitzGerald, Chn.
	B. M. Harrod
"Inst. Mining Engineers	Dr. Henry M. Howe
	John Hays Hammond
"Am. Chemical Socy	Prof. H. P. Talbot
	Dr. Clifford Richardson
"Inst. Electrical Engineers	Dr. Samuel Sheldon
	C. F. Scott
"Am. Socy. Mech. Engrs	Prof. Alex. C. Humphreys
	F. W. Taylor
"Society for Eng. Education	Prof. Dugald C. Jackson
	Prof. C. L. Crandall
	Dean James M. White
"Genl. Ed. Board	E. Benj. Andrews
"Carnegie Foundation	Henry S. Prichett

"This Committee has organized and have done me the honor to elect me Chairman.

"It was determined to begin the work of the Committee by making a study of the present methods of teaching Engineering and this work is now in progress.

"Very truly yours,
"DESMOND FITZGERALD."

Proposed Amendment to the Constitution.

THE PRESIDENT.—The next business is the proposed amendment to the Constitution. This amendment was offered by Mr. Clemens Herschel and other members of the Society, was considered by the Business Meeting of the Annual Convention in Denver, June 23d, 1908, and was in the following form:

Proposed Amendment to the Constitution (continued).

Amend Section 4 of Article III as follows: Instead of the word "twenty" in the fifth line thereof, insert the words "one-fifth"; and after the word "votes" in the same line, insert the words: "of all the votes cast". So that the new sentence shall read: "One-fifth or more negative votes of all the votes cast shall exclude from election."

This was amended by the Business Meeting of the Annual Convention to read as follows:

"Negative votes equal to one per cent., or to the whole number nearest to one per cent., of the total corporate membership at the time of voting shall exclude from election."

The amendment as amended was referred to a special committee to report at this Annual Meeting. The committee so appointed consists of Messrs. George H. Pegram, John W. Ellis and Henry R. Leonard.

MR. PEGRAM.—I hand you a report.

THE SECRETARY.—Do you want me to read it?

MR. PEGRAM.—If you please.

THE SECRETARY.—The report is as follows:

JANUARY 16TH, 1909.

Report of Committee on Proposed Amendment to the Constitution.

TO THE AMERICAN SOCIETY OF CIVIL ENGINEERS:

Your Committee, appointed pursuant to a resolution of the Business Meeting at Denver, June 23d, 1908, to consider an amendment to the Constitution proposed by Mr. Clemens Herschel and others, relating to the election of members, begs to report as follows:

The proposed amendment is:

"Amend Section 4 of Article III as follows:

"Instead of the word 'twenty' in the 5th line thereof insert the words 'One-fifth'; and after the word 'votes' in the same line, insert the words: 'of all the votes cast.' So that the new sentence shall read:

" 'One-fifth or more negative votes of all the votes cast shall exclude from election.' "

This was amended by the Denver Meeting by substituting, "One per cent. of the corporate membership", for, "One-fifth of all the votes cast".

The proposed amendment, together with two others relating to the same subject, was considered by a Committee which reported to the Denver Meeting at which the matter was discussed, and the majority report of the Committee together with the expressions of the views of the other members were printed and sent out with the letter-ballots on the amendment placing the election of members in the Board of Direction, which was adopted.

The merits of the amendments were so thoroughly considered in the report referred to, and the question has been so generally discussed, and apparently settled by the adoption of the amendment placing the election of members in the Board of Direction, that the duty of your Committee would seem to be to advise how the proposed amendment can be properly disposed of.

The purpose of this amendment was to correct a defect in the Constitution, which has been corrected, and it might appear that it is therefore void; but the Constitution requires that proposed amendments shall be voted upon by letter-ballot after having been con-

sidered by a general meeting which has power only to amend it in a manner pertinent to the original amendment, and if not amended it shall be voted upon by letter-ballot in the original form.

There would, therefore, seem to be no power to declare the proposed amendment void, and it would be a dangerous precedent to assume it so, because there might be proposed amendments which, while anticipated in part by amendments adopted since their presentation, would still have desirable features not affected by such action.

On the other hand, the discussions and action in this matter would indicate that the adoption of this proposed amendment is not desired or, in fact, advisable until the Constitution as amended October 7th, 1908, has had a period of trial.

Your Committee, therefore, recommends that the proposed amendment be re-worded by the Secretary to apply to the Constitution as it now exists, and sent out to letter-ballot with an accompanying statement that it must be voted upon by letter-ballot to meet the requirements of the Constitution; but that in view of the adoption of an amendment since its presentation placing the election of members in the hands of the Board of Direction, it is the opinion of this Committee that it should be defeated.

Respectfully submitted,
GEO. H. PEGRAM,
JOHN W. ELLIS,
H. R. LEONARD,
Committee.

J. A. BENSEL, M. AM. SOC. C. E.—I move that the Report of the Committee be received by the Society and its recommendations adopted.

THE PRESIDENT.—You have heard the motion, gentlemen. Is the motion seconded?

The motion was duly seconded.

THE PRESIDENT.—All in favor of that motion signify by saying "aye"; contrary, "no." It is a vote.

Announcements.

THE SECRETARY.—Mr. President, I take this opportunity of making an announcement in regard to the programme. This announcement has been made by placards, which are posted throughout the House, but somebody may not have seen them.

The excursion this afternoon is by automobile. The number of people who said they would come to this meeting—some 1 100—make it somewhat uncertain as to how many can be taken on that trip, and the limit of possibility in that direction is about 500 persons.

So it was determined to issue tickets, and to ask the resident members, who can at all times see the bridges, to wait until the non-residents have secured the tickets they need before applying for their own.

In addition to this, through the courtesy of the Commissioner of the Department of Charities, a steamer which will hold at least 250 people, will be in waiting at the foot of East Sixtieth Street, and will leave there at 3.45 this afternoon, sailing down the East River, and

Announcements (continued).

around the Battery, and up to the Chelsea Docks to meet the party that goes in automobiles.

I presume everybody has a copy of the programme of the meeting, and, as far as I know, there has been no change in the times of trains or in anything else. The special train to the Ashokan Reservoir, to-morrow, will stop at Newburg to take on passengers, and will also stop at Kingston. The time of arrival at Kingston is 12.05, and at Newburg about an hour earlier, so that those in that vicinity need not come to New York to take the special train.

I do not think I have anything else to say, Mr. President. I know of no other business at the present moment.

THE PRESIDENT.—If there is any new business to be considered we have ample time, as the ballot does not close until 12 o'clock.

Method of Appointing the Nominating Committee.

SAMUEL WHINERY, M. AM. SOC. C. E.—Mr. President, it has been evident for many years that the method now in vogue for choosing the Nominating Committee of the Society has worked unsatisfactorily. Perhaps no better illustration of that fact could be adduced than our proceedings here to-day. Out of a very large membership, the results, as read by the Secretary to-day, show not only that a very small part of the membership has taken any part whatever in this very important question of selecting the Nominating Committee, and that a similar small part, a mere fraction of the total membership of the Society, had any part in selecting the persons who have been elected here to-day as members of that Nominating Committee.

That has been the case for many years, and it seems to me it is a situation that demands the attention of the Society. I have taken some pains to collect the figures for the past six years, that is, from 1903 to 1908. It appears that the Nominating Committee of this Society, on whom depends the nomination of the officers of the Society—a most important function—that the members of the Nominating Committee have been elected upon the basis of $7\frac{2}{10}\%$ of the membership of the Society. In other words, $7\frac{2}{10}\%$ of the corporate membership of the Society, upon the average in the last five years, have dictated the election of the officers of this Society. It has appeared in that time that in a number of cases, as few as two (I won't say within this period as few as two, but previous to that I remember a case where only two members voted in a District) voted for the person who was officially elected as the member of the Nominating Committee.

It has often occurred that five or less members in this meeting have dictated the member of the Nominating Committee. It appears further—I might say to begin with that the method of electing the Nominating Committee of the Society is probably familiar to you, but the provisions of the Constitution are very short, and it will take but a moment to read them to you,

"At the Annual Meeting of each year, seven Corporate Members, not officers of the Society, one from each of the geographical districts, shall be appointed by the meeting to serve for two years; who, with the five living last Past-Presidents of the Society, shall be a committee to nominate officers for the Society.

"The Board of Direction may prescribe the mode of procedure for appointing this Committee.

"The Committee so appointed shall meet at the Annual Convention of the Society, and nominate candidates to fill the offices, named in Article V, so as to provide, with the officers holding over, a Vice-President and six Directors residing in District No. 1, and twelve Directors, divided equally, with regard to number and residence, among the remaining districts, Nos. 2, 3, 4, 5, 6 and 7."

The procedure has been that about two months before the Annual Meeting of the Society the Secretary sends out printed blanks to the Corporate Members, asking that the blanks shall be returned filled out with the member's suggestion of a name for member of the Nominating Committee. When these are returned the results are collated by the Secretary, as has been done to-day. It will appear that some one person has more votes than another usually. Occasionally two or three will have the same number, but usually one member has a larger number of votes; but be that number two, three, or four or ten, it has been the universal custom, I think, in this Society, with one or two exceptions—one of which occurred to-day—to elect that person as a member of the Nominating Committee who has the largest number of votes.

Now, that is certainly not the basis on which the American Society of Civil Engineers should proceed to elect the officers of the Society, and it seems to me that while the matter has worked fairly well so far, it is high time that the Society should make some change in that respect. There is no criticism to be offered with reference to the *personnel* or the work of past Nominating Committees. We have been very fortunate, I think, in that respect. That good fortune, however, is due to the high character of the men who have happened to be elected to the Nominating Committee—I say "happened", because that is precisely the fact—and to the good sense of the membership, and the absence in the Society of any disposition of selfishness or political action. To that alone has been due the fact that we have done as well as we have.

The facts show that 74 Corporate Members, in 1903, elected a majority of the one-half of the Nominating Committee elected that year. Of course, you understand that the Nominating Committee consists now of 19 persons, 14 of whom are elected. Of the 19, 10 form a majority, and would be able under any circumstances to control the nominations or the actions of the Committee.

Now, in 1903 only 74 Corporate Members effected the election of

Method of Appointing the Nominating Committee (continued).

five members of the Nominating Committee; in the next year 78 Corporate Members dictated the nomination of five members. So that about 75 members in those two years elected a controlling element in the Nominating Committee. In the next year, 1905, only 50 Corporate Members elected five of the seven members elected that year. The next year, 1906, was a more fortunate year, 120 then were required to elect the five members. In the next year, 1907, only 65 Corporate Members elected five members of the Committee.

It appears, therefore, from these figures, that if it were true in this Society, as it is in some others, that selfish methods or personal interests prevailed, or political methods, it would be entirely practicable for a bunch of say half a dozen members of the Society in the course of two years to secure absolute control of the Nominating Committee.

Now, while we have had no bad results from this in the past, and while there is no reason to anticipate that we shall in the future, still it is a condition of affairs that we ought to remedy. It ought not to be possible in the American Society of Civil Engineers for a bunch of less than one hundred men to dictate who shall be the officers of the Society, as is possible now.

It would not be possible for an Annual Meeting to proceed differently from what it does. The members act on the spur of the moment. There is no time for consideration or for conference as to the fitness of this or that person, and the Annual Meeting can only do practically as it has done to-day.

The trouble seems to lie with the apathy or the neglect of the members to reply to the circular letter of the Secretary. But there must be some cause antecedent to this. Surely on a matter of this importance more than 200 or 300 members will take sufficient interest in the welfare of this Society to fill out and sign a simple blank. The fact is, however, that these circular letters go to the members, many of whom are busy. They lay them aside. There is nothing to call their special attention to the matter or to the importance of it. In a great many cases they are never filled out, or only returned in very few cases. Is there in the districts any consideration, any means, any method or attempt to concentrate the views of the membership upon some one person, who would form a suitable member of the Nominating Committee? It seems to me that this is a defect in the present methods of the Society, and an important matter that we should take some means to remedy. That remedy, according to the Constitution, is placed in the hands of the Board of Direction.

I, therefore, Mr. President, offer the following preamble and resolution:

"*Whereas:* The present method of appointing the Nominating Committee of the Society has, in practice, proved to be unrepresentative and unsatisfactory, and

"*Whereas:* Section 2 of Article VII of the Constitution provides that the Board of Direction may prescribe the mode of procedure for appointing the Nominating Committee, therefore be it

"*Resolved:* That the Board of Direction be requested to give further consideration to the whole subject of the method of nominating officers for the Society; and particularly to the present mode of procedure in ascertaining and recording an expression of the corporate membership with regard to the choice of members of the Nominating Committee in the several geographical districts; and to report its conclusions and recommendations to the next Annual Convention of the Society."

Mr. President, I move the adoption of that resolution.

The motion was duly seconded.

THE PRESIDENT.—You have heard the motion, gentlemen. All in favor of it signify by saying "aye"; contrary, "no." It is a vote.

THE SECRETARY.—The Tellers are not ready to report, but they are very nearly ready, and I have no doubt from what I have seen that they will be ready about 12 o'clock.

THE PRESIDENT.—If there is no further business, it would be desirable to take a recess until the Tellers report.

GARDNER S. WILLIAMS, M. AM. SOC. C. E.—I move that we take a recess until five minutes of twelve.

The motion was duly seconded.

THE PRESIDENT.—You have heard the motion. All in favor of that signify it by saying "aye." Recess.

THE SECRETARY.—Before you rise, I would like to suggest the probability that we will have to serve luncheon to a number of you in this room, so that it will not be necessary for you to go down stairs.

After recess:

THE PRESIDENT.—The report of the Tellers will now be read. Ballot for Officers.

THE SECRETARY.—Mr. President, the Tellers have had a very hard struggle, and they have not been able to get their report in very good shape, but the results are here. The Tellers, Messrs. G. W. Parsons, S. C. Hulse, and Herbert E. Cantwell, report the total number of votes received and counted as 1 083.

For President:	
ONWARD BATES	1 072
Scattering	5
For Vice-Presidents:	
GEORGE H. PEGRAM	1 063
EMIL SWENSSON	1 043
Scattering	26
For Treasurer:	
JOSEPH M. KNAP	1 075
Scattering	1

Ballot for Officers (continued).

For Directors:

F. L. STUART	1 032
S. C. THOMPSON	1 046
W. G. WILKINS	1 041
A. N. TALBOT	1 038
W. M. GARDNER	1 044
H. A. SUMNER	1 042
Scattering	53

Officers Elected.

THE PRESIDENT.—In accordance with the vote which you have just heard read there have been elected for President, Onward Bates; Vice-Presidents, George H. Pegram and E. Swensson; for Treasurer, Joseph M. Knap; for Directors, F. L. Stuart, S. C. Thompson, W. G. Wilkins, A. N. Talbot, W. M. Gardner, H. A. Sumner; and I declare these gentlemen elected to the offices indicated.

I will ask Mr. Benzenberg and Mr. Noble if they will be good enough to escort the new President to the Chair.

MR. BENZENBERG.—I do not think Mr. Noble is in the room.

THE PRESIDENT.—Is Mr. Stearns here?

Mr. Bates was escorted to the Chair by Past-Presidents Benzenberg and Stearns.

THE PRESIDENT.—It is my great pleasure, sir, to announce your election as President for the ensuing year, and to present you with this gavel, and to wish you every success in your new position.

MR. BATES.—Gentlemen, and members of the American Society of Civil Engineers, an engineer should always be prepared to accept promotion to higher and better work. I am very well aware that the American Society of Civil Engineers in electing me its President has conferred on me its highest honor, but I prefer to think of it as an engagement for service. Therefore, I lay the honor on the table during my time of service, and will endeavor to fill the office and meet the trust which has been given to me by my brethren until I complete my term of office and have my name added to the list of honored Past-Presidents. I may then rejoice in the honor, which will be a lasting one.

This is an occasion for few words, and about all that I can add is that I fail to find words which seem to me to express properly my grateful appreciation.

THE SECRETARY.—Mr. President, I think it only remains to announce that a meeting of the Board of Direction will be held as soon as this meeting adjourns, and to ask all members of the Board of Direction if they will not come to the Committee Room, on the first floor, adjoining the Secretary's office at once, in order to get through with the business there is to be done.

MR. MACDONALD.—I move we adjourn.

The motion was duly seconded.

THE PRESIDENT.—Those in favor of adjournment will please say "aye"; contrary, "no." Carried.

Adjourned.

REPORTS OF SPECIAL COMMITTEES.

PROGRESS REPORT OF SPECIAL COMMITTEE ON UNIFORM TESTS OF CEMENT.

PRESENTED AT THE ANNUAL MEETING, JANUARY 20TH, 1909.

Your Committee on Uniform Tests of Cement has devoted much time, and given very careful consideration, to the subject. Frequent meetings have been held, and a number of investigations carried on, some of which cannot be finally reported at this time.

On several matters which have been considered, such as the substitution of a natural sand for the standard quartz, and the tests for normal consistency and constancy of volume, the Committee has not reached final conclusions.

While not prepared to submit a final report, the Committee feels that it should present a report of progress in order that the Society may be informed of the results of its investigations and conclusions.

The work of the Committee has been confined entirely to the methods for making tests, without attempting to specify what tests should be made.

In order to do full justice to the subject under consideration, it will be necessary to compile and report the results of the experiments now under way, and your Committee asks, therefore, that it be continued.

SAMPLING.

1.—*Selection of Sample.*—The selection of the sample for testing is a detail that must be left to the discretion of the engineer; the number and the quantity to be taken from each package will depend largely on the importance of the work, the number of tests to be made and the facilities for making them.

2.—The sample shall be a fair average of the contents of the package; it is recommended that, where conditions permit, one barrel in every ten be sampled.

3.—Samples should be passed through a sieve having twenty meshes per linear inch, in order to break up lumps and remove foreign material; this is also a very effective method for mixing them together in order to obtain an average. For determining the characteristics of a shipment of cement, the individual samples may be mixed and the average tested; where time will permit, however, it is recommended that they be tested separately.

4.—*Method of Sampling.*—Cement in barrels should be sampled through a hole made in the center of one of the staves, midway between

the heads, or in the head, by means of an auger or a sampling iron similar to that used by sugar inspectors. If in bags, it should be taken from surface to center.

Chemical Analysis.

5.—*Significance.*—Chemical analysis may render valuable service in the detection of adulteration of cement with considerable amounts of inert material, such as slag or ground limestone. It is of use, also, in determining whether certain constituents, believed to be harmful when in excess of a certain percentage, as magnesia and sulphuric anhydride, are present in inadmissible proportions.

6.—The determination of the principal constituents of cement—silica, alumina, iron oxide and lime—is not conclusive as an indication of quality. Faulty character of cement results more frequently from imperfect preparation of the raw material or defective burning than from incorrect proportions of the constituents. Cement made from very finely-ground material, and thoroughly burned, may contain much more lime than the amount usually present, and still be perfectly sound. On the other hand, cements low in lime may, on account of careless preparation of the raw material, be of dangerous character. Further, the ash of the fuel used in burning may so greatly modify the composition of the product as largely to destroy the significance of the results of analysis.

7.—*Method.*—As a method to be followed for the analysis of cement, that proposed by the Committee on Uniformity in the Analysis of Materials for the Portland Cement Industry, of the New York Section of the Society for Chemical Industry, and published in *Engineering News,* Vol. 50, p. 60, 1903; and in *The Engineering Record,* Vol. 48, p. 49, 1903, is recommended.

Specific Gravity.

8.—*Significance.*—The specific gravity of cement is lowered by adulteration and hydration, but the adulteration must be in considerable quantity to affect the results appreciably.

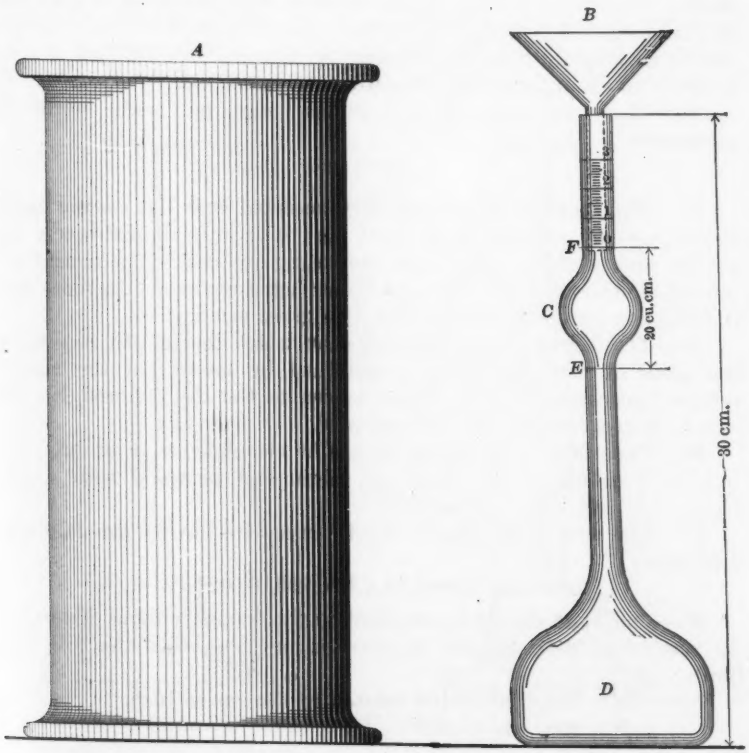
9.—Inasmuch as the differences in specific gravity are usually very small, great care must be exercised in making the determination.

10.—*Apparatus and Method.*—The determination of specific gravity is most conveniently made with Le Châtelier's apparatus. This consists of a flask (*D*), Fig. 1, of 120 cu. cm. (7.32 cu. in.) capacity, the neck of which is about 20 cm. (7.87 in.) long; in the middle of this neck is a bulb (*C*), above and below which are two marks (*F*) and (*E*); the volume between these marks is 20 cu. cm. (1.22 cu. in.). The neck has a diameter of about 9 mm. (0.35 in.), and is graduated into tenths of cubic centimeters above the mark (*F*).

11.—Benzine (62° Baumé naphtha), or kerosene free from water, should be used in making the determination.

12.—The specific gravity can be determined in two ways:

(1) The flask is filled with either of these liquids to the lower mark (*E*), and 64 g. (2.25 oz.) of powder, cooled to the temperature of the liquid, is gradually introduced through the funnel (*B*) [the stem of which extends into the flask to the top of the bulb (*C*)], until the upper mark (*F*) is reached. The difference in weight between the cement remaining and the original quantity (64 g.) is the weight which has displaced 20 cu. cm.



LE CHATELIER'S SPECIFIC GRAVITY APPARATUS.

FIG. 1.

13.—(2) The whole quantity of the powder is introduced, and the level of the liquid rises to some division of the graduated neck. This reading plus 20 cu. cm. is the volume displaced by 64 g. of the powder.

14.—The specific gravity is then obtained from the formula:

$$\text{Specific Gravity} = \frac{\text{Weight of Cement, in grammes,}}{\text{Displaced Volume, in cubic centimeters.}}$$

15.—The flask, during the operation, is kept immersed in water in a jar (*A*), in order to avoid variations in the temperature of the liquid. The results should agree within 0.01. The determination of specific gravity should be made on the cement as received; and, should it fall below 3.10, a second determination should be made on the sample ignited at a low red heat.

16.—A convenient method for cleaning the apparatus is as follows: The flask is inverted over a large vessel, preferably a glass jar, and shaken vertically until the liquid starts to flow freely; it is then held still in a vertical position until empty; the remaining traces of cement can be removed in a similar manner by pouring into the flask a small quantity of clean liquid and repeating the operation.

17.—More accurate determinations may be made with the picnometer.

Fineness.

18.—*Significance.*—It is generally accepted that the coarser particles in cement are practically inert, and it is only the extremely fine powder that possesses adhesive or cementing qualities. The more finely cement is pulverized, all other conditions being the same, the more sand it will carry and produce a mortar of a given strength.

19.—The degree of final pulverization which the cement receives at the place of manufacture is ascertained by measuring the residue retained on certain sieves. Those known as the No. 100 and No. 200 sieves are recommended for this purpose.

20.—*Apparatus.*—The sieves should be circular, about 20 cm. (7.87 in.) in diameter, 6 cm. (2.36 in.) high, and provided with a pan, 5 cm. (1.97 in.) deep, and a cover.

21.—The wire cloth should be of brass wire having the following diameters:

No. 100, 0.0045 in.; No. 200, 0.0024 in.

22.—This cloth should be mounted on the frames without distortion; the mesh should be regular in spacing and be within the following limits:

No. 100, 96 to 100 meshes to the linear inch.
No. 200, 188 to 200 " " " " "

23.—Fifty grammes (1.76 oz.) or 100 g. (3.52 oz.) should be used for the test, and dried at a temperature of 100° cent. (212° Fahr.) prior to sieving.

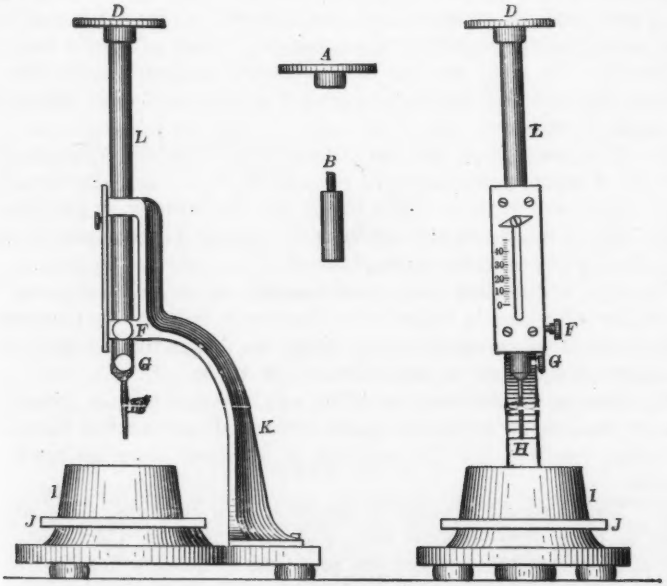
24.—*Method.*—The Committee, after careful investigation, has reached the conclusion that mechanical sieving is not as practicable or efficient as hand work, and therefore recommends the following method:

25.—The thoroughly dried and coarsely screened sample is weighed and placed on the No. 200 sieve, which, with pan and cover attached, is held in one hand in a slightly inclined position, and moved forward and backward, at the same time striking the side gently with the palm

of the other hand, at the rate of about 200 strokes per minute. The operation is continued until not more than one-tenth of 1% passes through after one minute of continuous sieving. The residue is weighed, then placed on the No. 100 sieve and the operation repeated. The work may be expedited by placing in the sieve a small quantity of large steel shot. The results should be reported to the nearest tenth of 1 per cent.

Normal Consistency.

26.—*Significance.*—The use of a proper percentage of water in making the pastes* from which pats, tests of setting, and briquettes are made, is exceedingly important, and affects vitally the results obtained.



VICAT NEEDLE.
Fig. 2.

27.—The determination consists in measuring the amount of water required to reduce the cement to a given state of plasticity, or to what is usually designated the normal consistency.

28.—Various methods have been proposed for making this determination, none of which has been found entirely satisfactory. The Committee recommends the following:

29.—*Method. Vicat Needle Apparatus.*—This consists of a frame (*K*), Fig. 2, bearing a movable rod (*L*), with the cap (*A*) at one end,

*The term "paste" is used in this report to designate a mixture of cement and water and the word "mortar" a mixture of cement, sand, and water.

and at the other the cylinder (*B*), 1 cm. (0.39 in.) in diameter, the cap, rod, and cylinder weighing 300 g. (10.58 oz.). The rod, which can be held in any desired position by a screw (*F*), carries an indicator, which moves over a scale (graduated to centimeters) attached to the frame (*K*). The paste is held by a conical, hard-rubber ring (*I*), 7 cm. (2.76 in.) in diameter at the base, 4 cm. (1.57 in.) high, resting on a glass plate (*J*), about 10 cm. (3.94 in.) square.

30.—In making the determination, the same quantity of cement as will be subsequently used for each batch in making the briquettes, but not less than 500 g., is kneaded into a paste, as described in Paragraph 56, and quickly formed into a ball with the hands, completing the operation by tossing it six times from one hand to the other, maintained 6 in. apart; the ball is then pressed into the rubber ring, through the larger opening, smoothed off, and placed (on its large end) on a glass plate and the smaller end smoothed off with a trowel; the paste, confined in the ring, resting on the plate, is placed under the rod bearing the cylinder, which is brought in contact with the surface and quickly released.

31.—The paste is of normal consistency when the cylinder penetrates to a point in the mass 10 mm. (0.39 in.) below the top of the ring. Great care must be taken to fill the ring exactly to the top.

32.—The trial pastes are made with varying percentages of water until the correct consistency is obtained.

33.—The Committee has recommended, as normal, a paste, the consistency of which is rather wet, because it believes that variations in the amount of compression to which the briquette is subjected in moulding are likely to be less with such a paste.

34.—Having determined in this manner the proper percentage of water required to produce a paste of normal consistency, the proper percentage required for the mortars is obtained from an empirical formula.

35.—The Committee hopes to devise such a formula. The subject proves to be a very difficult one, and, although the Committee has given it much study, it is not yet prepared to make a definite recommendation.

Time of Setting.

36.—*Significance.*—The object of this test is to determine the time which elapses from the moment water is added until the paste ceases to be fluid and plastic (called the "initial set"), and also the time required for it to acquire a certain degree of hardness (called the "final" or "hard set"). The former of these is the more important, since, with the commencement of setting, the process of crystallization or hardening is said to begin. As a disturbance of this process may produce a loss of strength, it is desirable to complete the operation of

mixing and moulding or incorporating the mortar into the work before the cement begins to set.

37.—It is usual to measure arbitrarily the beginning and end of the setting by the penetration of weighted wires of given diameters.

38.—*Method.*—For this purpose the Vicat Needle, which has already been described in Paragraph 29, should be used.

39.—In making the test, a paste of normal consistency is moulded and placed under the rod (*L*), Fig. 2, as described in Paragraph 30; this rod, bearing the cap (*D*) at one end and the needle (*H*), 1 mm. (0.039 in.) in diameter, at the other, weighing 300 g. (10.58 oz.). The needle is then carefully brought in contact with the surface of the paste and quickly released.

40.—The setting is said to have commenced when the needle ceases to pass a point 5 mm. (0.20 in.) above the upper surface of the glass plate, and is said to have terminated the moment the needle does not sink visibly into the mass.

41.—The test pieces should be stored in moist air during the test; this is accomplished by placing them on a rack over water contained in a pan and covered with a damp cloth, the cloth to be kept away from them by means of a wire screen; or they may be stored in a moist box or closet.

42.—Care should be taken to keep the needle clean, as the collection of cement on the sides of the needle retards the penetration, while cement on the point reduces the area and tends to increase the penetration.

43.—The determination of the time of setting is only approximate, being materially affected by the temperature of the mixing water, the temperature and humidity of the air during the test, the percentage of water used, and the amount of moulding the paste receives.

Standard Sand.

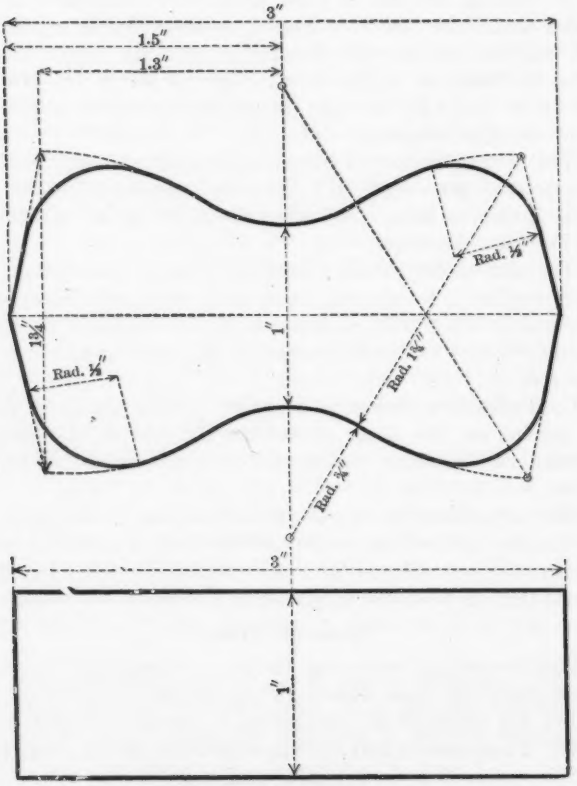
44.—The Committee recognizes the grave objections to the standard quartz now generally used, especially on account of its high percentage of voids, the difficulty of compacting in the moulds, and its lack of uniformity; it has spent much time in investigating the various natural sands which appeared to be available and suitable for use.

45.—For the present, the Committee recommends the natural sand from Ottawa, Ill., screened to pass a sieve having 20 meshes per linear inch and retained on a sieve having 30 meshes per linear inch; the wires to have diameters of 0.0165 and 0.0112 in., respectively, *i. e.*, half the width of the opening in each case. Sand having passed the No. 20 sieve shall be considered standard when not more than 1% passes a No. 30 sieve after one minute's continuous sifting of a 500-g. sample.*

* The Sandusky Portland Cement Company, of Sandusky, Ohio, has agreed to undertake the preparation of this sand, and to furnish it at a price only sufficient to cover the actual cost of preparation.

Form of Briquette.

46.—While the form of the briquette recommended by a former Committee of the Society is not wholly satisfactory, this Committee is not prepared to suggest any change, other than rounding off the corners by curves of ½-in. radius, Fig. 3.



DETAILS FOR BRIQUETTE.

Fig. 3.

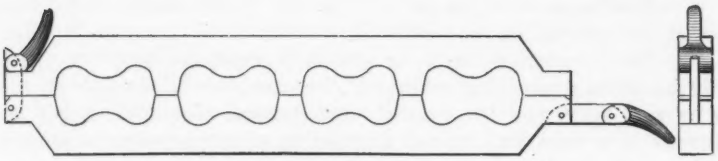
Moulds.

47.—The moulds should be made of brass, bronze, or some equally non-corrodible material, having sufficient metal in the sides to prevent spreading during moulding.

48.—Gang moulds, which permit moulding a number of briquettes at one time, are preferred by many to single moulds; since the greater

quantity of mortar that can be mixed tends to produce greater uniformity in the results. The type shown in Fig. 4 is recommended.

49.—The moulds should be wiped with an oily cloth before using.



DETAILS FOR GANG MOULD.

FIG. 4.

MIXING.

50.—All proportions should be stated by weight; the quantity of water to be used should be stated as a percentage of the dry material.

51.—The metric system is recommended because of the convenient relation of the gramme and the cubic centimeter.

52.—The temperature of the room and the mixing water should be as near 21° cent. (70° Fahr.) as it is practicable to maintain it.

53.—The sand and cement should be thoroughly mixed dry. The mixing should be done on some non-absorbing surface, preferably plate glass. If the mixing must be done on an absorbing surface it should be thoroughly dampened prior to use.

54.—The quantity of material to be mixed at one time depends on the number of test pieces to be made; about 1 000 g. (35.28 oz.) makes a convenient quantity to mix, especially by hand methods.

55.—The Committee, after investigation of the various mechanical mixing machines, has decided not to recommend any machine that has thus far been devised, for the following reasons:

(1) The tendency of most cement is to "ball up" in the machine, thereby preventing the working of it into a homogeneous paste; (2) there are no means of ascertaining when the mixing is complete without stopping the machine, and (3) the difficulty of keeping the machine clean.

56.—*Method.*—The material is weighed and placed on the mixing table, and a crater formed in the center, into which the proper percentage of clean water is poured; the material on the outer edge is turned into the crater by the aid of a trowel. As soon as the water has been absorbed, which should not require more than one minute, the operation is completed by vigorously kneading with the hands for an additional one minute, the process being similar to that used in kneading dough. A sand-glass affords a convenient guide for the time of

kneading. During the operation of mixing, the hands should be protected by gloves, preferably of rubber.

Moulding.

57.—Having worked the paste or mortar to the proper consistency, it is at once placed in the moulds by hand.

58.—The Committee has been unable to secure satisfactory results with the present moulding machines; the operation of machine moulding is very slow, and the present types permit of moulding but one briquette at a time, and are not practicable with the pastes or mortars herein recommended.

59.—*Method.*—The moulds should be filled immediately after the mixing is completed, the material pressed in firmly with the fingers and smoothed off with a trowel without mechanical ramming; the material should be heaped up on the upper surface of the mould, and, in smoothing off, the trowel should be drawn over the mould in such a manner as to exert a moderate pressure on the excess material. The mould should be turned over and the operation repeated.

60.—A check upon the uniformity of the mixing and moulding is afforded by weighing the briquettes just prior to immersion, or upon removal from the moist closet. Briquettes which vary in weight more than 3% from the average should not be tested.

Storage of the Test Pieces.

61.—During the first 24 hours after moulding, the test pieces should be kept in moist air to prevent them from drying out.

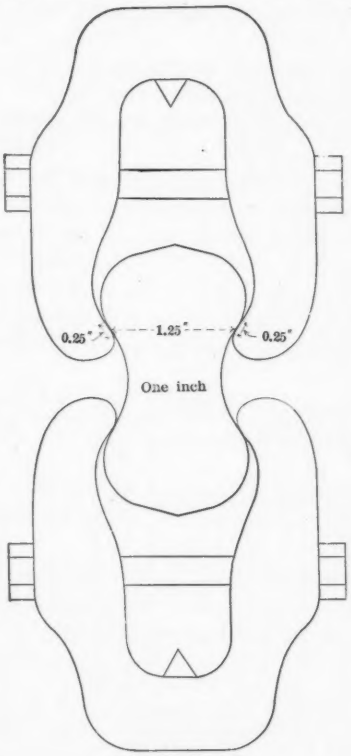
62.—A moist closet or chamber is so easily devised that the use of the damp cloth should be abandoned if possible. Covering the test pieces with a damp cloth is objectionable, as commonly used, because the cloth may dry out unequally, and, in consequence, the test pieces are not all maintained under the same condition. Where a moist closet is not available, a cloth may be used and kept uniformly wet by immersing the ends in water. It should be kept from direct contact with the test pieces by means of a wire screen or some similar arrangement.

63.—A moist closet consists of a soapstone or slate box, or a metal-lined wooden box—the metal lining being covered with felt and this felt kept wet. The bottom of the box is so constructed as to hold water, and the sides are provided with cleats for holding glass shelves on which to place the briquettes. Care should be taken to keep the air in the closet uniformly moist.

64.—After 24 hours in moist air, the test pieces for longer periods of time should be immersed in water maintained as near 21° cent. (70° Fahr.) as practicable; they may be stored in tanks or pans, which should be of non-corrodible material.

Tensile Strength.

65.—The tests may be made on any standard machine. A solid metal clip, as shown in Fig. 5, is recommended. This clip is to be used without cushioning at the points of contact with the test specimen. The bearing at each point of contact should be ¼ in. wide, and the distance between the center of contact on the same clip should be 1¼ in.



FORM OF CLIP.
Fig. 5.

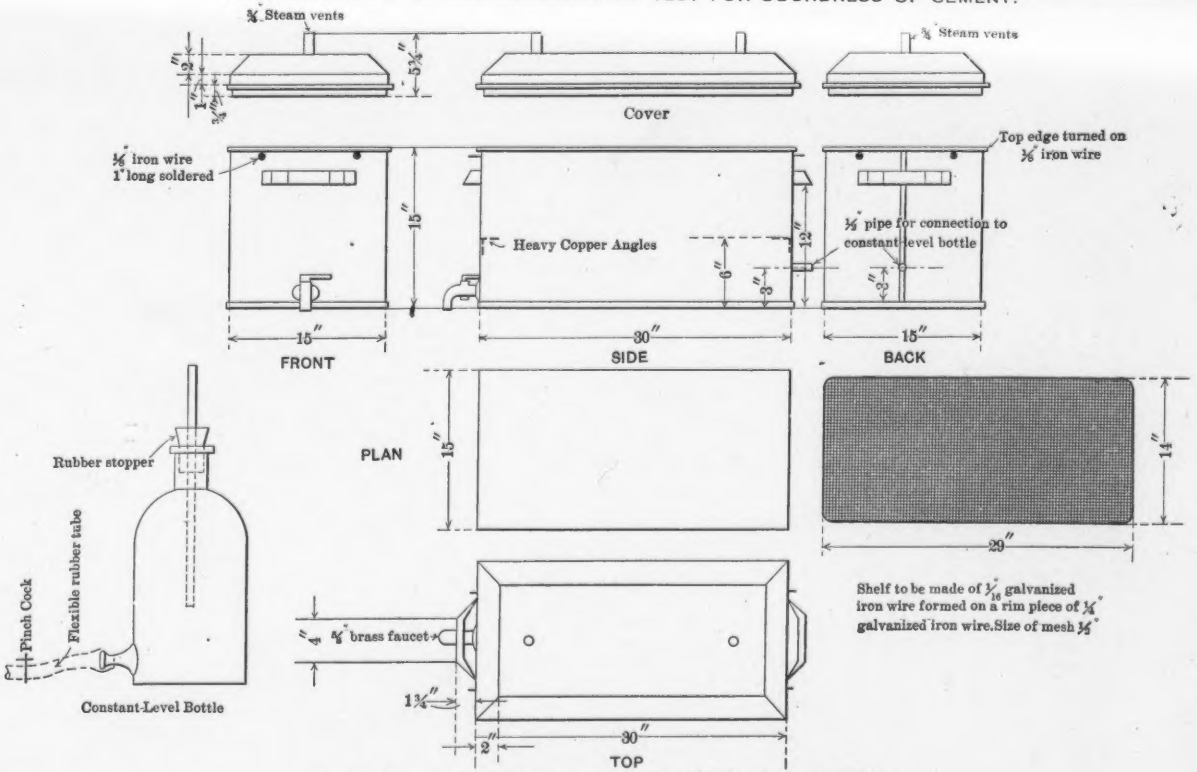
66.—Test pieces should be broken as soon as they are removed from the water. Care should be observed in centering the briquettes in the testing machine, as cross-strains, produced by improper centering, tend to lower the breaking strength. The load should not be applied too suddenly, as it may produce vibration, the shock from which often breaks the briquette before the ultimate strength is reached. Care must be taken that the clips and the sides of the briquette be clean and free from grains of sand or dirt, which would prevent a good bearing. The load should be applied at the rate of 600 lb. per min. The average of the briquettes of each sample tested should be taken as the test, excluding any results which are manifestly faulty.

Constancy of Volume.

67.—*Significance.*—The object is to develop those qualities which tend to destroy the strength and durability of a cement. As it is highly essential to determine such qualities at once, tests of this character are for the most part made in a very short time, and are known, therefore, as accelerated tests. Failure is revealed by cracking, checking, swelling, or disintegration, or all of these phenomena. A cement which remains perfectly sound is said to be of constant volume.

68.—*Methods.*—Tests for constancy of volume are divided into two classes: (1) normal tests, or those made in either air or water maintained at about 21° cent. (70° Fahr.), and (2) accelerated tests,

APPARATUS FOR MAKING ACCELERATED TEST FOR SOUNDNESS OF CEMENT.



Boiler to be made of sheet copper weighing 22 oz. per sq. ft., tinned inside.
All seams to be lapped where possible. Hard solder to be used only

FIG. 6.

or those made in air, steam, or water at a temperature of 45° cent. (113° Fahr.) and upward. The test pieces should be allowed to remain 24 hours in moist air before immersion in water or steam, or preservation in air.

69.—For these tests, pats, about 7½ cm. (2.95 in.) in diameter, 1¼ cm. (0.49 in.) thick at the center, and tapering to a thin edge, should be made, upon a clean glass plate [about 10 cm. (3.94 in.) square], from cement paste of normal consistency.

70.—*Normal Test.*—A pat is immersed in water maintained as near 21° cent. (70° Fahr.) as possible for 28 days, and observed at intervals. A similar pat, after 24 hours in moist air, is maintained in air at ordinary temperature and observed at intervals.

71.—*Accelerated Test.*—A pat is exposed in any convenient way in an atmosphere of steam, above boiling water, in a loosely closed vessel, for 5 hours. The apparatus recommended for making these determinations is shown in Fig. 6.

72.—To pass these tests satisfactorily, the pats should remain firm and hard, and show no signs of cracking, distortion or disintegration.

73.—Should the pat leave the plate, distortion may be detected best with a straight-edge applied to the surface which was in contact with the plate.

74.—In the present state of our knowledge it cannot be said that cement should necessarily be condemned simply for failure to pass the accelerated tests; nor can a cement be considered entirely satisfactory simply because it has passed these tests.

The above is a revision of the Progress Report of your Committee presented at the Annual Meeting, January 21st, 1903, and embodying all the changes which have since been suggested from time to time.

Matters under investigation, therein referred to, have been considered, as follows:

(1) The determination of the character, size, and shape of grains necessary for uniformity in Standard Sand;

(2) The form of briquette and clips to be used in tension tests, and the form of test pieces in compression tests;

(3) The proper method of making compression tests;

(4) The determination of the proper percentage of water for use in sand mixtures;

(5) The best method for the determination of soundness or constancy of volume.

APPARATUS FOR MAKING ACCELERATED TEST FOR SOUNDNESS OF CEMENT.

¾ Steam vents

The progress made has not been as great as was expected, and your Committee finds itself unable to present a report on the above named subjects at this time. In the matter of Standard Sand, it feels confident that it will be able to present a report at the next Annual Meeting, and asks to be continued.

Submitted on behalf of the Committee,

GEORGE S. WEBSTER,
Chairman.
RICHARD L. HUMPHREY,
Secretary.

DECEMBER 28TH, 1908.

Committee.
GEORGE S. WEBSTER,
RICHARD L. HUMPHREY,
GEORGE F. SWAIN,
ALFRED NOBLE,
LOUIS C. SABIN,
S. B. NEWBERRY,
CLIFFORD RICHARDSON,
W. B. W. HOWE,
F. H. LEWIS.

PROGRESS REPORT OF SPECIAL COMMITTEE ON CONCRETE AND REINFORCED CONCRETE.

PRESENTED TO THE ANNUAL MEETING, JANUARY 20TH, 1909.

"This Report has not yet been Placed Before or Accepted By the Society. Its Discussion has been made an Order of Business for the Next Annual Convention, July 6th-9th, 1909, and Written Communications concerning it Received Before that Date, will be Published in 'Proceedings'. All Rights to Publication are Reserved."

MAJORITY REPORT.*

I. INTRODUCTION.

Special Committees were appointed by the American Society of Civil Engineers, American Society for Testing Materials, American Railway Engineering and Maintenance of Way Association, and the Association of American Portland Cement Manufacturers, for the purpose of investigating current practice and providing definite information concerning the properties of concrete and reinforced concrete and to recommend necessary factors and formulas required in the design of structures in which these materials are used.

At the Annual Convention of the American Society of Civil Engineers held at Asheville, N. C., June 11th, 1903, the following resolution was adopted:

It is the sense of this meeting that a Special Committee be appointed to take up the question of concrete and steel-concrete, and that such committee co-operate with the American Society for Testing Materials, and the American Railway Engineering and Maintenance of Way Association.

Following the adoption of this resolution, a Special Committee on concrete and steel-concrete was appointed by the Board of Direction on May 31st, 1904. At the Annual Meeting, held January 18th, 1905, the title of this special committee was, at the request of the Committee, changed to "Special Committee on Concrete and Reinforced Concrete."

At the annual meeting of the American Society for Testing Materials held July 1st, 1903, at the Delaware Water Gap, the following resolution was unanimously adopted:

That the Executive Committee be requested to consider the desirability of appointing a committee on "Reinforced Concrete," with a view of co-operating with the committees of other societies in the study of the subject.

At the meeting of the Executive Committee of that Society, held December 5th, 1903, a special committee on "Reinforced Concrete" was appointed.

* For Minority Report, see page 119.

The American Railway Engineering and Maintenance of Way Association appointed a Committee on Masonry on July 20th, 1899, with instructions, as a part of its duties, to prepare specifications for concrete masonry. A preliminary set of specifications for Portland cement concrete was reported to and adopted by the Association on March 19th, 1903. At the meeting held in Chicago on March 17th, 1904, the Committee on Masonry was authorized to co-operate with the Special Committee on Concrete and Reinforced Concrete of the American Society of Civil Engineers, and following this action a special subcommittee was appointed.

At a meeting of the several special committees representing the above-mentioned societies, held at Atlantic City, N. J., June 17th, 1904, arrangements were completed for collaborating the work of these several committees through the formation of the Joint Committee on Concrete and Reinforced Concrete. Mr. C. C. Schneider was elected temporary chairman and Professor A. N. Talbot was elected temporary secretary. The proposed plan of action of the special committee of the American Society of Civil Engineers was outlined, involving the appointment of subcommittees on Plan and Scope, on Tests, and on Ways and Means.

Mr. Schneider and Mr. Schaub, as Chairman and Secretary of the Special Committee of the American Society of Civil Engineers, were elected permanent Chairman and Secretary of the Joint Committee on Concrete and Reinforced Concrete. Mr. Emil Swensson was elected Vice-Chairman, and, on the resignation of Mr. Schaub, Mr. Richard L. Humphrey was elected Secretary.

The following is the organization of the Joint Committee:

OFFICERS.

Chairman—C. C. Schneider.
Vice-Chairman—Emil Swensson.
Secretary—Richard L. Humphrey.

MEMBERS.

American Society of Civil Engineers (Special Committee on Concrete and Reinforced Concrete):

Greiner, J. E., Consulting Engineer, Baltimore and Ohio Railroad, Baltimore, Md.

Hatt, W. K., Professor of Civil Engineering, Purdue University, Lafayette, Ind.

Hoff, Olaf, Vice-President, Butler Brothers, Hoff and Company, New York, N. Y.

Humphrey, Richard L., Consulting Engineer, Engineer in Charge, Structural Materials Testing Laboratories, U. S. Geological Survey, Philadelphia, Pa.

Lesley, R. W., President, American Cement Company, Philadelphia, Pa.

Schaub, J. W., Consulting Engineer, Chicago, Ill.

Schneider, C. C., Consulting Engineer, Philadelphia, Pa.
Swensson, Emil, Consulting Engineer, Pittsburg, Pa.
Talbot, A. N., Professor of Municipal and Sanitary Engineering, Charge of Theoretical and Applied Mechanics, University of Illinois, Urbana, Ill.
Worcester, J. R., Consulting Engineer, Boston, Mass.

American Society for Testing Materials (Committee on Reinforced Concrete):

Fuller, William B., Consulting Engineer, New York, N. Y.
Heidenreich, E. Lee, Consulting Engineer, New York, N. Y.
Humphrey, Richard L., Consulting Engineer, Engineer in Charge, Structural Materials Testing Laboratories, U. S. Geological Survey, Philadelphia, Pa.
Johnson, Albert L., Consulting Engineer, St. Louis, Mo.
Lanza, Gaetano, Professor of Theoretical and Applied Mechanics, Massachusetts Institute of Technology, Boston, Mass.
Lesley, R. W., President, American Cement Company, Philadelphia, Pa.
Marburg, Edgar, Professor of Civil Engineering, University of Pennsylvania, Philadelphia, Pa.
Mills, Charles M., Principal Assistant Engineer, Philadelphia Rapid Transit Company, Philadelphia, Pa.
Moisseiff, Leon S., Assistant Engineer, Department of Bridges, New York, N. Y.
Quimby, Henry H., Assistant Engineer of Bridges, Bureau of Surveys, Philadelphia, Pa.
Taylor, W. P., Engineer in Charge of Testing Laboratory, Philadelphia, Pa.
Thompson, Sanford E., Consulting Engineer, Newton Highlands, Mass.
Turneaure, F. E., Dean of College of Mechanics and Engineering, University of Wisconsin, Madison, Wis.
Wagner, Samuel Tobias, Assistant Engineer, Philadelphia and Reading Railroad, Philadelphia, Pa.
Webster, George S., Chief Engineer, Bureau of Surveys, Philadelphia, Pa.

American Railway Engineering and Maintenance of Way Association (Subcommittee on Reinforced Concrete):

Boynton, C. W., Inspecting Engineer, Universal Portland Cement Company, Chicago, Ill.
Cunningham, A. O., Chief Engineer, Wabash Railroad, St. Louis, Mo.
Moore, C. H., Engineer of Grade Crossings, Erie Railroad, New York, N. Y.
Scribner, Gilbert H., Jr., Contracting Engineer, Chicago, Ill.
Swain, George F., Professor of Civil Engineering, Massachusetts Institute of Technology, Boston, Mass.

Association of American Portland Cement Manufacturers (Committee on Concrete and Steel Concrete):

Fraser, Norman D., President, Chicago Portland Cement Company, Chicago, Ill.

Griffiths, R. E., Vice-President, American Cement Company, Philadelphia, Pa.
Hagar, Edward M., President, Universal Portland Cement Company, Chicago, Ill.
Newberry, Spencer B., Manager, Sandusky Portland Cement Company, Sandusky, Ohio.

The report, herein presented, embodies the present judgment of the Committee concerning the proper use of Concrete and Reinforced Concrete.

II. Adaptability of Concrete and Reinforced Concrete.

The adaptability of concrete and reinforced concrete for engineering structures, or parts thereof, is now so well established that it may be considered one of the recognized materials of construction. It has proved to be a satisfactory material, when properly used, for those purposes for which its qualities make it particularly suitable.

1. Proper Use.

Concrete is a material of very low tensile strength and capable of sustaining but very small tensile deformations without rupture; its value as a structural material depends chiefly upon its durability, its fire-resisting qualities, its strength in compression and its relatively low cost. Its strength increases generally with age.

Plain concrete or massive concrete is well adapted for structural forms in which the principal stresses are compressive. These include foundations, dams, retaining and other walls, piers, abutments, short columns and, in many cases, arches. In the design of massive concrete, the tensile strength of the material must generally be neglected.

By the use of metal reinforcement to resist the principal tensile stresses, concrete becomes available for general use in a great variety of structures and structural forms. This combination of concrete and metal is particularly advantageous in the beam, where both compression and tension exist; it is also advantageous in the column, where the main stresses are compressive, but where cross-bending may exist. In structures resisting lateral forces it possesses advantages over plain concrete in that it may be so designed as to utilize more fully the strength rather than the weight of the material.

2. Improper Use.

Failures of reinforced concrete structures are usually due to any one or a combination of the following causes: defective design, poor material, and faulty execution.

The defects in a design may be many and various. The computations and assumptions on which they are based may be faulty and contrary to the established principles of statics and mechanics; the unit stresses used may be excessive, or the details of the design defective.

The design of reinforced concrete structures should receive at least the same careful consideration as those of steel, and only engineers with sufficient experience and good judgment should be intrusted with such work.

The computations should include all minor details, which are sometimes of the utmost importance. The design should show clearly the size and position of the reinforcement, and should provide for proper connections between the component parts, so that they cannot be displaced. As the connections between reinforced concrete members are frequently a source of weakness, the design should include a detailed study of such connections, accompanied by computations to prove their strength.

The use of high unit stresses, approaching the danger line, is a defect in the design of reinforced concrete structures.

Articulated concrete structures, designed in imitation of steel trusses, may be mentioned as illustrating a questionable use of reinforced concrete.

Poor material is sometimes used for the concrete, as well as for the reinforcement. The use of inferior concrete is generally due to a lack of experience of the contractor and his superintendents, or to the absence of proper supervision.

An unsuitable quality of steel for reinforcement is sometimes prescribed in specifications, for the purpose of reducing the cost. For steel structures, a high grade of material is specified, while the steel used for reinforcing concrete is sometimes made of unsuitable, brittle material.

Faulty execution and careless workmanship may generally be attributed to unintelligent or insufficient supervision.

While other engineering structures, upon the safety of which human lives depend, are generally designed by engineers employed by the owner, and the contracts let on the engineer's design and specifications, in accordance with legitimate practice, reinforced concrete structures frequently are designed by contractors or by engineers commercially interested, and the contract let for a lump sum.

The construction of buildings in large cities is regulated by ordinances or building laws, and the work is inspected by municipal authorities. For reinforced concrete work, however, the limited supervision which municipal inspectors are able to give is not sufficient. Means for more adequate supervision and inspection should, therefore, be provided.

3. Responsibility and Supervision.

The execution of the work should not be separated from the design, since intelligent supervision and successful execution can be expected only when both functions are combined. The engineer who prepares the design and specifications should therefore have the supervision of the execution of the work.

The Committee recommends the following rules for structures of reinforced concrete, for the purpose of fixing the responsibility and providing for adequate supervision during construction:

a. Before work is commenced, complete plans shall be prepared, accompanied by specifications, static computations and descriptions showing the general arrangement and all details. The static computations shall give the loads assumed separately, such as dead and live loads, wind and impact, if any, and the resulting stresses.

b. The specifications shall state the qualities of the materials to be used for making the concrete, and the manner in which they are to be proportioned.

c. The strength which the concrete is expected to attain after a definite period shall be stated in the specifications.

d. The drawings and specifications shall be signed by the engineer and the contractor.

e. The approval of plans and specifications by other authorities shall not relieve the engineer nor the contractor of responsibility.

f. Inspection during construction shall be made by competent inspectors employed by, and under the supervision of, the engineer, and shall cover the following:

1. The materials.
2. The correct construction and erection of the forms and the supports.
3. The sizes, shapes and arrangement of the reinforcement.
4. The proportioning, mixing and placing of the concrete.
5. The strength of the concrete by tests of standard test pieces made on the work.
6. Whether the concrete is sufficiently hardened before the forms and supports are removed.
7. Prevention of injury to any part of the structure by and after the removal of the forms.
8. Comparison of dimensions of all parts of the finished structure with the plans.

g. Load tests on portions of the finished structure shall be made where there is reasonable suspicion that the work has not been properly performed, or that, through influences of some kind, the strength has been impaired. Loading shall be carried to such a point that twice the calculated working stresses in critical parts are reached, and such loads shall cause no permanent deformations. Load tests shall not be made until after 60 days of hardening.

4. Destructive Agencies.

a. Corrosion of Metal Reinforcement.—Tests and experience have proved that steel embedded in good concrete will not corrode, no matter whether located above or below fresh or sea water level. If the

concrete is porous, so as to be readily permeable to water, as, where the concrete is laid with a very dry consistency, the metal may be corroded in the presence of moisture.

b. Electrolysis.—There is little accurate information available as to the effect of electrolysis on concrete. The few experiments that are available seem to indicate that concrete may be damaged through the leakage of small electrical currents through the mass, particularly where steel is embedded in the concrete. These experiments are not conclusive, however, and the large numbers of reinforced concrete structures subject to the action of electrolysis, in which the metal and concrete are in perfect condition, would seem to indicate that the destructive action reported was due to abnormal conditions which do not often occur in practice.

c. Salt Water.—The data available concerning the effect of sea water on concrete or reinforced concrete are inconclusive and limited in amount. There have been no authentic cases reported where the disintegration has proved to be due entirely to sea water. The decomposition that has been reported manifests itself in a number of ways; in some cases the mortar softens and crumbles; in others, a crust forms which in time comes off. It has been found, however, that where concrete is proportioned in such a way as to secure a maximum density and is mixed thoroughly it makes an impervious concrete, upon which sea water has apparently little effect. Sea-walls have been standing for considerable lengths of time without apparent injury. In many of our harbors where the water has been rendered brackish through the rivers discharging into them, the action that has been reported has been at the water line and was probably due in part to freezing.

d. Acids.—Concrete of first-class quality, thoroughly hardened, is affected appreciably only by strong acids which seriously injure other materials. A substance like manure, because of the acid in its composition, is injurious to green concrete, but after the concrete has thoroughly hardened it satisfactorily resists such action.

e. Oils.—When concrete is properly made and the surface carefully finished and hardened it resists the action of such oils as petroleum and ordinary engine oils. Certain oils which contain fatty acids appear to produce injurious effects.

f. Alkalies.—The action of alkalies on concrete is problematical. In the reclamation of arid land, where the soil is heavily charged with alkaline salts, it has been found that concrete, stone, brick, iron and other materials are injured under certain conditions. It would seem that at the level of the ground-water such structures are disintegrated, possibly due in part to the effect of formation of crystals resulting from the alternate wetting and drying of the surface of the concrete at this ground-water line. Such destructive action can be prevented by the use of an insulating coating.

III. Materials.

A knowledge of the properties of the materials entering into concrete and reinforced concrete is the first essential. The importance of the quality of the materials used cannot be over-estimated, and, not only the cement, but also the aggregates, should be subject to such definite requirements and tests as will insure a concrete of the required quality.

1. Cement.

There are available, for construction purposes, Portland, Natural, and Puzzolan or Slag cements. Only Portland cement is suitable for reinforced concrete.

a. Portland Cement is the finely pulverized product resulting from the calcination to incipient fusion of an intimate mixture of properly proportioned argillaceous and calcareous materials. It has a definite chemical composition varying within comparatively narrow limits.

Portland cement should be used in reinforced concrete construction and any construction that will be subject to shocks or vibrations or stresses other than direct compression.

b. Natural Cement is the finely pulverized product resulting from the calcination of an argillaceous limestone at a temperature only sufficient to drive off the carbonic acid gas. While the limestone must have a certain composition, this composition may vary in much wider limits than in the case of Portland cement. Natural cement does not develop its strength as quickly nor is it as uniform in composition as Portland cement.

Natural cement may be used in massive masonry where weight rather than strength is the essential feature.

Where economy is the governing factor, a comparison may be made between the use of Natural cement and a leaner mixture of Portland cement that will develop the same strength.

c. Puzzolan or Slag Cement is the finely pulverized product resulting from grinding a mechanical mixture of granulated basic blast-furnace slag and hydrated lime.

Puzzolan cement is not nearly as strong, uniform, or reliable as Portland or Natural cement, is not extensively used, and never in important work; it should be used only for foundation work underground where it is not exposed to air or running water.

d. Specifications. The cement should meet the requirements of the Standard Specifications for Cement (see Appendix, p. 108). A number of societies have been working on methods for testing and specifications for cement. The best practice seems to be represented in the standard methods of testing and specifications for cement which are the result of the joint labors of Special Committees of the American Society of Civil Engineers, American Society for Testing Materials,

American Railway Engineering and Maintenance of Way Association, American Institute of Architects, and others.

2. Aggregates.

Extreme care should be exercised in selecting the aggregates for mortar and concrete, and careful tests made of the materials for the purpose of determining their qualities and the grading necessary to secure maximum density* or a minimum percentage of voids.

a. Fine Aggregate consists of sand, crushed stone, or gravel screenings, passing when dry a screen having ¼-in. diameter holes. It should be preferably of silicious material, clean, coarse, free from vegetable loam or other deleterious matter.

A gradation of the grain from fine to coarse is generally advantageous.

Mortars composed of one part Portland cement and three parts fine aggregate by weight when made into briquettes should show a tensile strength of at least 70% of the strength of 1:3 mortar of the same consistency made with the same cement and standard Ottawa sand.

b. Coarse Aggregate consists of inert material, such as crushed stone, or gravel, which is retained on a screen having ¼-in. diameter holes. The particles should be clean, hard, durable, and free from all deleterious material. Aggregates containing soft, flat or elongated particles should be excluded from important structures. A gradation of sizes of the particles is generally advantageous.

The maximum size of the coarse aggregate shall be such that it will not separate from the mortar in laying and will not prevent the concrete from fully surrounding the reinforcement and filling all parts of the forms. Where concrete is used in mass, the size of the coarse aggregate may be such as to pass a 3-in. ring. For reinforced members a size to pass a 1-in. ring, or a smaller size, may be used.

Cinder concrete is not suitable for reinforced concrete structures, and may be safely used only in mass for very light loads or for fireproofing.

Where cinder concrete is permissible, the cinders used as the coarse aggregate should be composed of hard, clean, vitreous clinker, free from sulphides, unburned coal, or ashes.

3. Water.

The water used in mixing concrete should be free from oil, acid, strong alkalies, or vegetable matter.

* A convenient coefficient of density is the ratio of the sum of the volumes of materials contained in a unit volume to the total unit volume.

4. Metal Reinforcement.

The Committee recommends, as a suitable material for reinforcement, steel filling the requirements of the specifications adopted by the American Railway Engineering and Maintenance of Way Association (Appendix, p. 112).

For the reinforcement of slabs, small beams, or minor details, or for the prevention of shrinkage cracks where wire or small rods are suitable, material conforming to the requirements of either Specification A or B given in the Appendix (pages 113 and 114) may be used.

The reinforcement should be free from rust, scale, or coatings of any character which would tend to reduce or destroy the bond.

IV. Preparation and Placing of Mortar and Concrete.

1. Proportions.

The materials to be used in concrete should be carefully selected, of uniform quality, and proportioned with a view to securing as nearly as possible a maximum density.

a. Unit of Measure.—The unit of measure should be the barrel, which should be taken as containing 3.8 cu. ft. Four bags containing 94 lb. of cement each should be considered the equivalent of one barrel. Fine and coarse aggregate should be measured separately as loosely thrown into the measuring receptacle.

b. Relation of Fine and Coarse Aggregates.—The fine and coarse aggregates should be used in such relative proportions as will insure maximum density. In unimportant work it is sufficient to do this by individual judgment, using correspondingly higher proportions of cement; for important work these proportions should be carefully determined by density experiments, and the sizing of the fine and coarse aggregates should be uniformly maintained, or the proportions changed to meet the varying sizes.

c. Relation of Cement and Aggregates.—For reinforced concrete construction, a density proportion based on 1:6 should generally be used, *i. e.*, one part of cement to a total of six parts of fine and coarse aggregates measured separately.

In columns, richer mixtures are often required, while for massive masonry or rubble concrete a leaner mixture, of 1:9 or even 1:12, may be used. These proportions should be determined by the strength or wearing qualities required in the construction at the critical period of its use. Experienced judgment based on individual observation and tests of similar conditions in similar localities is the best guide as to the proper proportions for any particular case.

2. Mixing.

The ingredients of concrete should be thoroughly mixed to the desired consistency, and the mixing should continue until the cement

is uniformly distributed and the mass is uniform in color and homogeneous, since the maximum density and therefore the greatest strength of a given mixture depends largely on thorough and complete mixing.

a. Measuring Ingredients.—Methods of measurement of the proportions of the various ingredients, including the water, should be used, which will secure separate uniform measurements at all times.

b. Machine Mixing.—When the conditions will permit, a machine mixer of a type which insures the uniform proportioning of the materials throughout the mass should be used, since a more thorough and uniform consistency can be thus obtained.

c. Hand Mixing.—When it is necessary to mix by hand, the mixing should be on a water-tight platform, and especial precautions should be taken to turn the materials until they are homogeneous in appearance and color.

d. Consistency.—The materials should be mixed wet enough to produce a concrete of such a consistency as will flow into the forms and about the metal reinforcement, and, at the same time, can be conveyed from the mixer to the forms without separation of the coarse aggregate from the mortar.

e. Retempering.—Retempering mortar or concrete, *i. e.,* remixing with water after it has partially set, should not be permitted.

3. Placing of Concrete.

a. Methods.—Concrete, after the addition of water to the mix, should be handled rapidly, and in as small masses as is practicable, from the place of mixing to the place of final deposit, and under no circumstances should concrete be used that has partially set before final placing. A slow-setting cement should be used when a long time is likely to occur between mixing and final placing.

The concrete should be deposited in such a manner as will permit the most thorough compacting, such as can be obtained by working with a straight shovel or slicing tool kept moving up and down until all the ingredients have settled in their proper place by gravity and the surplus water has been forced to the surface.

In depositing the concrete under water, special care should be exercised to prevent the cement from being floated away, and to prevent the formation of laitance which hardens very slowly and forms a poor surface on which to deposit fresh concrete. Laitance is formed in both still and running water, and should be removed before placing fresh concrete.

Before placing the concrete, care should be taken to see that the forms are substantial and thoroughly wetted and the space to be occupied by the concrete is free from débris. When the placing of the concrete is suspended, all necessary grooves for joining future work should be made before the concrete has had time to set.

When work is resumed, concrete previously placed should be roughened, thoroughly cleansed of foreign material and laitance, drenched and slushed with a mortar consisting of one part Portland cement and not more than two parts fine aggregate.

The faces of concrete exposed to premature drying should be kept wet for a period of at least seven days.

b. Freezing Weather.—Concrete for reinforced structures should not be mixed or deposited at a freezing temperature, unless special precautions are taken to avoid the use of materials containing frost or covered with ice crystals, and to provide means to prevent the concrete from freezing after being placed in position and until it has thoroughly hardened.

c. Rubble Concrete.—Where the concrete is to be deposited in massive work, its value may be improved and its cost materially reduced through the use of clean stones thoroughly embedded in the concrete as near together as is possible and still entirely surrounded by concrete.

V. Forms.

Forms should be substantial and unyielding, so that the concrete shall conform to the designed dimensions and contours, and should be tight to prevent the leakage of mortar.

The time for removal of forms is one of the most important steps in the erection of a structure of concrete or reinforced concrete. Care should be taken to inspect the concrete and ascertain its hardness before removing the forms.

So many conditions affect the hardening of concrete that the proper time for the removal of the forms should be decided by some competent and responsible person, especially where the atmospheric conditions are unfavorable.

VI. Details of Construction.

1. Joints.

a. Reinforcement.—Wherever in tension reinforcement it is necessary to splice the reinforcing bars, the length of lap shall be determined on the basis of the safe bond stress and the stress in the bar at the point of splice; or a connection shall be made between the bars of sufficient strength to carry the stress. Splices at points of maximum stress should be avoided. In columns, large bars should be properly butted and spliced; small bars may be treated as indicated for tension reinforcement, or their stress may be taken off by being embedded in large masses of concrete. At foundations, bearing plates should be provided for large bars or structural forms.

b. Concrete.—For concrete construction it is desirable to cast the entire structure at one operation, but as this is not always possible,

especially in large structures, it is necessary to stop the work at some convenient point. This point should be selected so that the resulting joint may have the least possible effect on the strength of the structure. It is therefore recommended that the joint in columns be made flush with the lower side of the girders; that the joints in girders be at a point midway between supports, but should a beam intersect a girder at this point, the joint should be offset a distance equal to twice the width of the beam; that the joints in the members of a floor system should in general be made at or near the center of the span.

Joints in columns should be perpendicular to the axis of the column, and in girders, beams and floor slabs perpendicular to the plane of their surfaces.

2. Shrinkage.

Girders should never be constructed over freshly formed columns without permitting a period of at least two hours to elapse, thus providing for settlement or shrinkage in the columns. Before resuming work, the top of the column should be thoroughly cleansed of foreign matter and laitance. If the concrete in the column has become hard, the top should also be drenched and slushed with a mortar consisting of one part Portland cement and not more than two parts fine aggregate before placing additional concrete.

3. Temperature Changes.

Concrete is sensitive to temperature changes, and it is necessary to take this fact into account in designing and erecting concrete structures. In some positions the concrete is subjected to a much greater fluctuation in temperature than in others, and in such cases joints are necessary. The frequency of these joints will depend, first, upon the range of temperature to which concrete will be subjected; second, upon the quantity and position of the reinforcement. These points should be determined and provided for in the design. In massive work, such as retaining walls, abutments, etc., built without reinforcement, joints should be provided, approximately, every 50 ft. throughout the length of the structure. To provide against the structures being thrown out of line by unequal settlement, each section of the wall may be tongued and grooved into the adjoining section. To provide against unsightly cracks, due to unequal settlement, a joint should be made at all sharp angles.

4. Fire-Proofing.

The actual fire tests of concrete and reinforced concrete have been limited, but experience, together with the results of tests so far made, indicate that concrete may be safely used for fire-proofing purposes. Concrete itself is incombustible and reasonably proof against fire when

composed of a silicious sand and a hard coarse aggregate such as igneous rock.

For a fire-proof covering, these same materials may be used, or clean hard-burned cinders may be substituted for the coarse aggregate.

The low rate of heat conductivity of concrete is one reason of its value for fire-proofing. The dehydration of the water of crystalization of concrete probably begins at about 500° Fahr., and is completed at about 900° Fahr., but experience indicates that the volatilization of the water absorbs heat from the surrounding mass, which, together with the resistance of the air cells, tends to increase the heat resistance of the concrete, so that the process of dehydration is very much retarded. The concrete that is actually affected by fire remains in position and affords protection to the concrete beneath it.

It is recommended that in monolithic concrete columns, the concrete to a depth of 1½ in. be considered as protective covering and not included in the effective section.

The thickness of the protective coating required depends upon the probable duration of a fire which is likely to occur in the structure, and should be based on the rate of heat conductivity. The question of the conductivity of concrete is one which requires further study and investigation before a definite rate for different classes of concrete can be fully established. However, for ordinary conditions it is recommended that the metal in girders and columns be protected by a minimum of 2 in. of concrete; that the metal in beams be protected by a minimum of 1½ in. of concrete, and that the metal in floor slabs be protected by a minimum of 1 in. of concrete.

It is recommended that the corners of columns, girders and beams be beveled or rounded, as a sharp corner is more seriously affected by fire than a round one.

5. Water-Proofing.

Many expedients have been used to render concrete impervious to water under normal conditions, and also under pressure conditions that exist in reservoirs, dams, and conduits of various kinds. Experience shows, however, that where mortar or concrete is proportioned to obtain the greatest practicable density and is mixed to a rather wet consistency, the resulting mortar or concrete is impervious under ordinary conditions. A concrete of dry consistency is more or less pervious to water, and compounds of various kinds have been mixed with the concrete, or applied as a wash to the surface for the purpose of making it water-tight. Many of these compounds are of but temporary value, and in time lose their power of imparting impermeability to the concrete.

In the case of subways, long retaining walls, and reservoirs, leakage cracks may be prevented by horizontal and vertical reinforcement,

properly proportioned and located, provided the concrete itself is impervious.

Such reinforcement distributes the stretch due to contraction or settlement so that the cracks are too minute to permit leakage, or are soon closed by infiltration of silt.

Asphaltic or coal-tar preparations, applied either as a mastic or as a coating on felt or cloth fabric, are used for water-proofing, and should be proof against injury by liquids or gases.

6. Surface Finish.

Concrete is a material of an individual type, and should not be used in imitation of other structural materials. One of the important problems connected with the use of concrete is the character of the finish of exposed surfaces. The finish of the surface should be determined before the concrete is placed, and the work conducted so as to make possible the finish desired. For many forms of construction the natural surface of the concrete is unobjectionable, but frequently the marks of the boards and the flat dead surface are displeasing, and make some special treatment desirable. A treatment of the surface which removes the film of mortar and brings the coarser particles of the concrete into relief is frequently used to remove the form markings, break the monotonous appearance of the surface, and make it more pleasing. Plastering of surfaces should be avoided, for the other methods of treatment are more reliable and usually much more satisfactory. Plastering, even if carefully applied, is likely to peel off under the action of frost or temperature changes.

VII. Design.

1. Massive Concrete.

In the design of massive concrete or plain concrete, no account should be taken of the tensile strength of the material, and sections should usually be so proportioned as to avoid tensile stresses. This will generally be accomplished, in the case of rectangular shapes, if the line of pressure is kept within the middle third of the section, but in very large structures, such as high masonry dams, a more exact analysis may be required. Structures of massive concrete are able to resist unbalanced lateral forces by reason of their weight, hence the element of weight rather than strength often determines the design. A relatively cheap and weak concrete will therefore often be suitable for massive concrete structures. Owing to its low extensibility, the contraction due to hardening and to temperature changes requires special consideration, and, except in the case of very massive walls such as dams, it is desirable to provide joints at intervals to localize the effect of such contrac-

tion. The spacing of such joints will depend upon the form and dimensions of the structure and its degree of exposure.

Massive concrete may well be used for piers and short columns, in which the ratio of length to least width is relatively small. Under ordinary conditions this ratio should not exceed six, but, where the central application of the load is assured, a somewhat higher value may safely be used.

Massive concrete is also a suitable material for arches of moderate span where the conditions as to foundations are favorable.

2. Reinforced Concrete.

By the use of metal reinforcement to resist the principal tensile stresses, concrete becomes available for general use in a great variety of structures and structural forms. This combination of concrete and steel is particularly advantageous in the beam, where both compression and tension exist; it is also advantageous in the column, where the main stresses are compressive, but where cross-bending may exist. The theory of design will therefore relate mainly to the analysis of beams and columns.

3. General Assumptions.

a. Loads.—The loads or forces to be resisted consist of:

1. The dead load, which includes the weight of the structure and fixed loads and forces.
2. The live load, or the loads and forces which are variable. The dynamic effect of the live load will often require consideration. Any allowance for the dynamic effect is preferably taken into account by adding the desired amount to the live load or to the live-load stresses. The working stresses hereinafter recommended are intended to apply to the equivalent static stresses so determined.

   In the case of high buildings the live load on columns may be reduced in accordance with the usual practice.

b. Lengths of Beams and Columns.—The span length for beams and slabs shall be taken as the distance from center to center of supports, but shall not be taken to exceed the clear span plus the depth of beam or slab. Brackets shall not be considered as reducing the clear span in the sense here intended.

The length of columns shall be taken as the maximum unsupported length.

c. Internal Stresses.—As a basis for calculations relating to the strength of structures, the following assumptions are recommended:

1. Calculations should be made with reference to working stresses and safe loads rather than with reference to ultimate strength and ultimate loads.

2. A plane section before bending remains plane after bending.
3. The modulus of elasticity of concrete in compression, within the usual limits of working stresses, is constant. The distribution of compressive stresses in beams is therefore rectilinear.
4. In calculating the moment of resistance of beams the tensile stresses in the concrete shall be neglected.
5. Perfect adhesion is assumed between concrete and reinforcement. Under compressive stresses the two materials are therefore stressed in proportion to their moduli of elasticity.
6. The ratio of the modulus of elasticity of steel to the modulus of elasticity of concrete may be taken at 15.
7. Initial stress in the reinforcement due to contraction or expansion in the concrete may be neglected.

It is appreciated that the assumptions herein given are not entirely borne out by experimental data. They are given in the interest of simplicity and uniformity, and variations from exact conditions are taken into account in the selection of formulas and working stresses.

For calculations relative to deflections, the tensile strength of the concrete should be taken into account. For such calculations, also, a value of 8 to 12 for the ratio of the moduli corresponds more nearly to the actual conditions and may well be used.

4. Tee-Beams.

In beam and slab construction, an effective bond should be provided at the junction of the beam and slab. When the principal slab reinforcement is parallel to the beam, transverse reinforcement should be used extending over the beam and well into the slab.

Where adequate bond between slab and web of beam is provided, the slab may be considered as an integral part of the beam, but its effective width shall be determined by the following rules:

a. It shall not exceed one-fourth of the span length of the beam;
b. Its overhanging width on either side of the web shall not exceed 4 times the thickness of the slab.

In the design of T-beams acting as continuous beams, due consideration should be given to the compressive stresses at the support.

5. Floor Slabs.

Floor slabs should be designed and reinforced as continuous over the supports. If the length of the slab exceeds 1.5 times its width

the entire load should be carried by transverse reinforcement. Square slabs may well be reinforced in both directions.*

The loads carried to beams by slabs which are reinforced in two directions will not be uniformly distributed to the supporting beam, and may be assumed to vary in accordance with the ordinates of a triangle. The moments in the beams should be calculated accordingly.

6. Continuous Beams and Slabs.

When the beam or slab is continuous over its supports, reinforcement should be fully provided at points of negative moment. In computing the positive and negative moments in beams and slabs continuous over several supports, due to uniformly distributed loads, the following rules are recommended:

a. That for floor slabs the bending moments at center and at support be taken at $\frac{w\,l^2}{12}$ for both dead and live loads, where w represents the load per linear foot and l the span length.

b. That for beams the bending moment at center and at support for interior spans be taken at $\frac{w\,l^2}{12}$, and for end spans it be taken at $\frac{w\,l^2}{10}$ for center and adjoining support for both dead and live loads.

In the case of beams and slabs continuous for two spans only, or of spans of unusual length, more exact calculations should be made. Special consideration is also required in the case of concentrated loads.

Where beams are reinforced on the compression side, the steel may be assumed to carry its proportion of stress in accordance with

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* The exact distribution of load on square and rectangular slabs, supported on four sides and reinforced in both directions cannot readily be determined. The following method of calculation is recognized to be faulty, but it is offered as a tentative method which will give results on the safe side. The distribution of load is first to be determined by the formula

$$r = \frac{l^4}{l^4 + b^4}$$

in which r = proportion of load carried by the transverse reinforcement, l = length and b = breadth of slab. For various ratios of l/b the values of r are as follows:

l/b	r
1	0.50
1.1	0.59
1.2	0.67
1.3	0.75
1.4	0.80
1.5	0.83

Using the values above specified, each set of reinforcement is to be calculated in the same manner as slabs having supports on two sides only, but the total amount of reinforcement thus determined may be reduced 25% by gradually increasing the rod spacing from the third point to the edge of the slab.

the provisions of Chap. VII, Section 3, *c*, paragraph 6. In the case of continuous beams, tensile and compressive reinforcement over supports must extend sufficiently beyond the support to develop the requisite bond strength.

7. Bond Strength and Spacing of Bars.

Adequate bond strength should be provided in accordance with the formula hereinafter given. Where a portion of the bars is bent up near the end of a beam, the bond stress in the remaining straight bars will be less than is represented by the theoretical formula.

Where high bond resistance is required, the deformed bar is a suitable means of supplying the necessary strength. Adequate bond strength throughout the length of a bar is preferable to end anchorage, but such anchorage may properly be used in special cases. Anchorage furnished by short bends at a right angle is less effective than hooks consisting of turns through 180 degrees.

The lateral spacing of parallel bars should not be less than two and one-half diameters, center to center, nor should the distance from the side of the beam to the center of the nearest bar be less than two diameters. The clear spacing between two layers of bars should not be less than $\frac{1}{2}$ in.

8. Shear and Diagonal Tension.

Calculations for web resistance shall be made on the basis of maximum shearing stress as determined by the formulas hereinafter given. When the maximum shearing stresses exceed the value allowed for the concrete alone, web reinforcement must be provided to aid in carrying the diagonal tension stresses. This web reinforcement may consist of bent bars, or inclined or vertical members attached to or looped about the horizontal reinforcement. Where inclined members are used, the connection to the horizontal reinforcement shall be such as to insure against slip.

Experiments bearing on the design of details of web reinforcement are not yet complete enough to allow more than general and tentative recommendations to be made. It is well established, however, that a very moderate amount of reinforcement, such as is furnished by a few bars bent up at small inclination, increases the strength of a beam against failure by diagonal tension to a considerable degree; and that a sufficient amount of web reinforcement can readily be provided to increase the shearing resistance to a value from three or more times that found when the bars are all horizontal and no web reinforcement is used. The following allowable values for the maximum shearing stress are therefore recommended, based on the working stresses of Section VIII, page 106.

a. For beams with horizontal bars, only 40 lb. per sq. in.

b. For beams in which a part of the horizontal reinforcement is used in the form of bent-up bars, arranged with due respect to the shearing stresses, a higher value may be allowed, but not to exceed 60 lb. per sq. in.

c. For beams thoroughly reinforced for shear, a value not exceeding 120 lb. per sq. in.

In the calculation of web reinforcement to provide the strength required under *c* above, the concrete may be counted upon as carrying one-third of the shear. The remainder is to be provided for by means of metal reinforcement consisting of bent rods or stirrups, but preferably both. The requisite amount of such reinforcement may be estimated on the assumption that the entire shear on a section, less the amount assumed to be carried by the concrete, is carried by the reinforcement in a length of beam equal to its depth.

The longitudinal spacing of stirrups or bent rods shall not exceed three-fourths the depth of the beam.

It is important that adequate bond strength be provided to develop fully the assumed strength of all shear reinforcement.

Inasmuch as small deformations in the horizontal steel tends to prevent the formation of diagonal cracks, a beam will be strengthened against diagonal tension failure by so arranging the horizontal reinforcement that the unit stresses at points of large shear shall be relatively low.

9. Columns.

It is recommended that the ratio of the unsupported length of a column to its least width be limited to 15.

The effective area of the column shall be taken as the area within the protective covering, as defined in Chap. VI, Section 4; or, in the case of hooped columns or columns reinforced with structural shapes, it shall be taken as the area within the hooping or structural shapes.

Columns may be reinforced by means of longitudinal rods, by bands or hoops, by bands or hoops together with longitudinal bars, or by structural forms which in themselves are sufficiently rigid to act as columns. The general effect of bands or hoops is to increase greatly the "toughness" of the column and its ultimate strength, but hooping has little effect upon its behavior within the limit of elasticity. It thus renders the concrete a safer and more reliable material, and should permit the use of a somewhat higher working stress. The beneficial effects of "toughening" are adequately provided by a moderate amount of hooping, a larger amount serving mainly to increase the ultimate strength and the possible deformation before ultimate failure.

The following recommendations are made for the relative working stresses in the concrete for the several types of columns:

a. Columns with longitudinal reinforcement only, the unit stress recommended for axial compression in Chap. VIII. Section 3.

b. Columns with reinforcement of bands or hoops, as hereinafter specified, stresses 20% higher than given for *a.*

c. Columns reinforced with not less than 1% and not more than 4% of longitudinal bars and with bands or hoops, stresses 45% higher than given for *a.*

d. Columns reinforced with structural steel column units which thoroughly encase the concrete core, stresses 45% higher than given for *a.*

In all cases, longitudinal steel is assumed to carry its proportion of stress in accordance with Section 3. The hoops or bands are not to be counted upon directly as adding to the strength of the column.

Bars composing longitudinal reinforcement shall be straight, and shall have sufficient lateral support to be securely held in place until the concrete has set.

Where bands or hoops are used, the total amount of such reinforcement shall not be less than 1% of the volume of the column disclosed. The clear spacing of such bands or hoops shall not be greater than one-fourth the diameter of the enclosed column. Adequate means must be provided to hold bands or hoops in place so as to form a column, the core of which shall be straight and well centered.

Bending stresses due to eccentric loads must be provided for by increasing the section until the maximum stress does not exceed the values above specified.

10. Reinforcing for Shrinkage and Temperature Stresses.

Where large areas of concrete are exposed to atmospheric conditions, the changes of form due to shrinkage (resulting from hardening) and to action of temperature are such that large cracks will occur in the mass, unless precautions are taken to so distribute the stresses as either to prevent the cracks altogether or to render them very small. The size of the cracks will be directly proportional to the diameter of the reinforcing bars and inversely proportional to the percentage of reinforcement and also to its bond resistance per unit of surface area. To be most effective, therefore, reinforcement should be placed near the surface and well distributed, and a form of reinforcement used which will develop a high bond resistance.

VIII. Working Stresses.

1. General Assumptions.

The following working stresses are recommended for static loads. Proper allowances for vibration and impact are to be added to live loads where necessary to produce an equivalent static load before applying the unit stresses in proportioning parts.

In selecting the permissible working stress to be allowed on concrete, we should be guided by the working stresses usually allowed for other materials of construction, so that all structures of the same class but composed of different materials may have approximately the same degree of safety.

The stresses for concrete are proposed for concrete composed of one part Portland cement and six parts of aggregates, capable of developing an average compressive strength of 2 000 lb. per sq. in. at 28 days, when tested in cylinders 8 in. in diameter and 16 in. long, under laboratory conditions of manufacture and storage, using the same consistency as is used in the field. In considering the factors recommended with relation to this strength, it is to be borne in mind that the strength at 28 days is by no means the ultimate which will be developed at a longer period, and therefore they do not correspond with the real factor of safety. On concretes, in which the material of the aggregate is inferior, all stresses should be proportionally reduced, and similar reduction should be made when leaner mixes are to be used. On the other hand, if, with the best quality of aggregates, the richness is increased, an increase may be made in all working stresses proportional to the increase in compressive strength at 28 days, but this increase shall not exceed 25 per cent.

2. Bearing.

When compression is applied to a surface of concrete larger than the loaded area, a stress of 32.5% of the compressive strength at 28 days, or 650 lb. per sq. in. on the above-described concrete, may be allowed. This pressure is probably unnecessarily low when the ratio of the stressed area to the whole area of the concrete is much below unity, but is recommended for general use rather than a variable unit based upon this ratio.

3. Axial Compression.

For concentric compression on a plain concrete column or pier, the length of which does not exceed 12 diameters, 22.50% of the compressive strength at 28 days, or 450 lb. per sq. in. on 2 000-lb. concrete, may be allowed.

For other forms of columns, the stresses obtained from the ratios given in Chapter VII, Section 9, may govern.

4. Compression in Extreme Fiber.

The extreme fiber stress of a beam, calculated on the assumption of a constant modulus of elasticity for concrete under working stresses, may be allowed to reach 32.5% of the compressive strength at 28 days, or 650 lb. per sq. in. for 2 000-lb. concrete. Adjacent to the support of continuous beams, stresses 15% higher may be used.

5. Shear and Diagonal Tension.

Where pure shearing stress occurs, that is, uncombined with compression normal to the shearing surface, and with all tension normal to the shearing plane provided for reinforcement, a shearing stress of 6% of the compressive strength at 28 days, or 120 lb. per sq. in. on 2 000-lb. concrete, may be allowed. Where the shear is combined with an equal compression, as on a section of a column at 45° with the axis, the stress may equal one-half the compressive stress allowed. For ratios of compressive stress to shear intermediate between 0 and 1, proportionate shearing stresses shall be used.

In calculations on beams in which diagonal tension is considered to be taken by the concrete, the vertical shearing stresses should not exceed 2% of the compressive strength at 28 days, or 40 lb. per sq. in. for 2 000-lb. concrete.

6. Bond.

The bonding stress between concrete and plain reinforcing bars may be assumed at 4% of the compressive strength at 28 days, or 80 lb. per sq. in. for 2 000-lb. concrete; in the case of drawn wire, 2% or 40 lb. on 2 000-lb. concrete.

7. Reinforcement.

The tensile stress in steel should not exceed 16 000 lb. per sq. in. The compressive stress in reinforcing steel should not exceed 16 000 lb. per sq. in., or 15 times the working compressive stress in the concrete.

In structural steel members, the working stresses adopted by the American Railway Engineering and Maintenance of Way Association are recommended.

8. Modulus of Elasticity.

The value of the modulus of elasticity of concrete has a wide range, depending upon the materials used, the age, the range of stresses between which it is considered, as well as other conditions. It is recommended that in all computations it be assumed as one-fifteenth that of steel, as, while not rigorously accurate, this assumption will give safe results.

JANUARY, 1909.

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APPENDIX.

1.

STANDARD SPECIFICATIONS.

1.

Standard Specifications for Cement.

Adopted August 15th, 1908, by the
American Society for Testing Materials.

General Observations.

1. These remarks have been prepared with a view of pointing out the pertinent features of the various requirements and the precautions to be observed in the interpretation of the results of the tests.

2. The Committee would suggest that the acceptance or rejection under these specifications be based on tests made by an experienced person having the proper means for making the tests.

SPECIFIC GRAVITY.

3. Specific gravity is useful in detecting adulteration. The results of tests of specific gravity are not necessarily conclusive as an indication of the quality of a cement, but when in combination with the results of other tests may afford valuable indications.

FINENESS.

4. The sieves should be kept thoroughly dry.

TIME OF SETTING.

5. Great care should be exercised to maintain the test pieces under as uniform conditions as possible. A sudden change or wide range of temperature in the room in which the tests are made, a very dry or humid atmosphere, and other irregularities, vitally affect the rate of setting.

TENSILE STRENGTH.

6. Each consumer must fix the minimum requirements for tensile strength to suit his own conditions. They shall, however, be within the limits stated.

CONSTANCY OF VOLUME.

7. The tests for constancy of volume are divided into two classes, the first normal, the second accelerated. The latter should be regarded as a precautionary test only, and not infallible. So many conditions enter into the making and interpreting of it that it should be used with extreme care.

8. In making the pats the greatest care should be exercised to avoid

initial strains due to molding or to too rapid drying-out during the first twenty-four hours. The pats should be preserved under the most uniform conditions possible, and rapid changes of temperature should be avoided.

9. The failure to meet the requirements of the accelerated tests need not be sufficient cause for rejection. The cement may, however, be held for twenty-eight days, and a retest made at the end of that period using a new sample. Failure to meet the requirements at this time should be considered sufficient cause for rejection, although in the present state of our knowledge it cannot be said that such failure necessarily indicates unsoundness, nor can the cement be considered entirely satisfactory simply because it passes the tests.

SPECIFICATIONS.

General Conditions.

1. All cement shall be inspected.

2. Cement may be inspected either at the place of manufacture or on the work.

3. In order to allow ample time for inspecting and testing, the cement should be stored in a suitable weather-tight building having the floor properly blocked or raised from the ground.

4. The cement shall be stored in such a manner as to permit easy access for proper inspection and identification of each shipment.

5. Every facility shall be provided by the Contractor and a period of at least twelve days allowed for the inspection and necessary tests.

6. Cement shall be delivered in suitable packages with the brand and name of manufacturer plainly marked thereon.

7. A bag of cement shall contain 94 pounds of cement net. Each barrel of Portland cement shall contain 4 bags, and each barrel of natural cement shall contain 3 bags of the above net weight.

8. Cement failing to meet the seven-day requirements may be held awaiting the results of the twenty-eight day tests before rejection.

9. All tests shall be made in accordance with the methods proposed by the Committee on Uniform Tests of Cement of the American Society of Civil Engineers, presented to the Society January 21, 1903, and amended January 20, 1904, and January 15, 1908, with all subsequent amendments thereto. (See addendum to these specifications.)

10. The acceptance or rejection shall be based on the following requirements:

Natural Cement.

11. *Definition.* This term shall be applied to the finely pulverized product resulting from the calcination of an argillaceous limestone at a temperature only sufficient to drive off the carbonic acid gas.

FINENESS.

12. It shall leave by weight a residue of not more than 10 per cent. on the No. 100, and 30 per cent. on the No. 200 sieve.

TIME OF SETTING.

13. It shall not develop initial set in less than ten minutes, and shall not develop hard set in less than thirty minutes, or in more than three hours.

TENSILE STRENGTH.

14. The minimum requirements for tensile strength for briquettes one inch square in cross section shall be within the following limits, and shall show no retrogression in strength within the periods specified.*

Age.	*Neat Cement.*	*Strength.*
24 hours in moist air		50-100 lbs.
7 days (1 day in moist air, 6 days in water)		100-200 "
28 days (1 " " 27 " ")		200-300 "

One Part Cement, Three Parts Standard Sand.

7 days (1 day in moist air, 6 days in water)	25- 75 "
28 days (1 " " 27 " ")	75-150 "

If the minimum strength is not specified, the mean of the above values shall be taken as the minimum strength required.

CONSTANCY OF VOLUME.

15. Pats of neat cement about three inches in diameter, one-half inch thick at center, tapering to a thin edge, shall be kept in moist air for a period of twenty-four hours.

(*a*) A pat is then kept in air at normal temperature.

(*b*) Another is kept in water maintained as near 70° F. as practicable.

16. These pats are observed at intervals for at least 28 days, and to satisfactorily pass the tests, should remain firm and hard and show no signs of distortion, checking, cracking or disintegrating.

PORTLAND CEMENT.

17. *Definition.* This term is applied to the finely pulverized product resulting from the calcination to incipient fusion of an intimate mixture of properly proportioned argillaceous and calcareous materials, and to which no addition greater than 3 per cent. has been made subsequent to calcination.

*For example, the minimum requirement for the twenty-four hour neat cement test should be some specified value within the limits of 50 and 100 pounds, and so on for each period stated.

SPECIFIC GRAVITY.

18. The specific gravity of the cement, ignited at a low red heat, shall be not less than 3.10, and the cement shall not show a loss on ignition of more than 4 per cent.

FINENESS.

19. It shall leave by weight a residue of not more than 8 per cent. on the No. 100, and not more than 25 per cent. on the No. 200 sieve.

TIME OF SETTING.

20. It shall not develop initial set in less than thirty minutes, and must develop hard set in not less than one hour, nor more than ten hours.

TENSILE STRENGTH.

21. The minimum requirements for tensile strength for briquettes one inch square in section shall be within the following limits, and shall show no retrogression in strength within the periods specified.*

Age.	*Neat Cement.*	*Strength.*
24 hours in moist air		150-200 lbs.
7 days (1 day in moist air, 6 days in water)		450-550 "
28 days (1 " " 27 " ")		550-650 "
	One Part Cement, Three Parts Standard Sand.	
7 days (1 day in moist air, 6 days in water)		150-200 lbs.
28 days (1 " " 27 " ")		200-300 "

If the minimum strength is not specified, the mean of the above values shall be taken as the minimum strength required.

CONSTANCY OF VOLUME.

22. Pats of neat cement about three inches in diameter, one-half inch thick at the center, and tapering to a thin edge, shall be kept in moist air for a period of twenty-four hours.

(*a*) A pat is then kept in air at normal temperature and observed at intervals for at least 28 days.

(*b*) Another pat is kept in water maintained as near 70° F. as practicable, and observed at intervals for at least 28 days.

(*c*) A third pat is exposed in any convenient way in an atmosphere of steam, above boiling water, in a loosely closed vessel for five hours.

23. These pats, to satisfactorily pass the requirements, shall remain firm and hard and show no signs of distortion, checking, cracking or disintegrating.

* For example, the minimum requirement for the twenty-four hour neat cement test should be some specified value within the limits of 150 and 200 pounds, and so on for each period stated.

SULPHURIC ACID AND MAGNESIA.

24. The cement shall not contain more than 1.75 per cent. of anhydrous sulphuric acid (SO_3), nor more than 4 per cent. of magnesia (MgO).

2.

1. Abstract Embodying Portions Applicable to Concrete Reinforcement from the Standard Specifications for Structural Steel.

Adopted March, 1906, by the
American Railway Engineering and Maintenance of Way Association.

Process of Manufacture.

83. Steel shall be made by the open-hearth process.

Schedule of Requirements.

84. The chemical and physical properties shall conform to the following limits:

Elements Considered.	Structural Steel.	Rivet Steel.
Phosphorus, max.... Basic	0.04 per cent.	0.04 per cent.
Acid	0.08 "	0.04 "
Sulphur, maximum	0.05 "	0.04 "
Ultimate tensile strength. Pounds per square inch	Desired 60 000	Desired 50 000
Elongation, minimum, percentage in 8 in.	1 500 000*	1 500 000
Fig. 1	Ult. tensile strength	Ult. tensile strength
Character of fracture	Silky	Silky
Cold bends without fracture	180° flat†	180° flat‡

* See paragraph 11. † See paragraphs 12, 13 and 14. ‡ See paragraph 15.

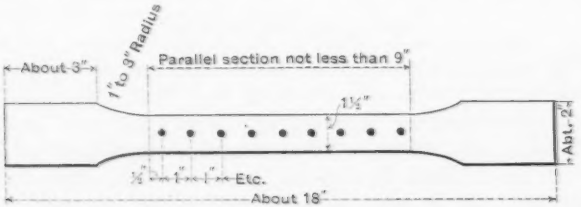


FIG. 1.

The yield point, as indicated by the drop of beam, shall be recorded in the test reports.

Allowable Variations.

85. If the ultimate strength varies more than 4 000 lb. from that desired, a retest shall be made on the same gauge, which, to be acceptable, shall be within 5 000 lb. of the desired ultimate.

Chemical Analyses.

86. Chemical determinations of the percentages of carbon, phosphorus, sulphur, and manganese shall be made by the manufacturer

from a test ingot taken at the time of the pouring of each melt of steel, and a correct copy of such analysis shall be furnished to the engineer or his inspector. Check analyses shall be made from finished material, if called for by the purchaser, in which case an excess of 25% above the required limits will be allowed.

Number of Tests.

92. At least one tensile and one bending test shall be made from each melt of steel as rolled.

Thick Material.

95. Full-sized material for steel 1 in. thick and over, tested as rolled, shall bend, cold, 180° around a pin, the diameter of which is equal to twice the thickness of the bar, without fracture on the outside of bend.

Finish.

98. Finished material shall be free from injurious seams, flaws, cracks, defective edges or other defects, and have a smooth, uniform and workmanlike finish. Plates 36 in. in width and under shall have rolled edges.

Stamping.

99. Every finished piece of steel shall have the melt number and the name of the manufacturer stamped or rolled upon it.

Defective Material.

100. Material which, subsequent to the above tests at the mills, and its acceptance there, develops weak spots, brittleness, cracks, or other imperfections, or is found to have injurious defects, will be rejected at the shop and shall be replaced by the manufacturer at his own cost.

3.

Alternative Specifications for Metal Reinforcement.

A.

1. For wire and rod reinforcements, the steel may be manufactured from Bessemer billets (not re-rolled rails) and shall meet the following requirements:

2. *Tensile Tests.*

Tensile strength, in pounds per square inch, not less than 105 000.

Yield point, in pounds per square inch, not less than 52 500 nor more than 90% of the tensile strength.

Elongation in 8 in., not less than 10%, with the following modifications:

a. For each increase in diameter of $\frac{1}{8}$ in. above $\frac{3}{4}$ in. a deduction of 1% shall be made from the specified elongation.

b. For materials from $\frac{1}{4}$ in. to, but not including, $\frac{5}{16}$ in. diameter the elongation shall be 8 per cent.

For material $\frac{5}{16}$ in. to $\frac{1}{4}$ in., elongation 7 per cent.

For material $\frac{3}{8}$ in. to $\frac{5}{16}$ in., elongation 6 per cent.

For material less than $\frac{3}{8}$ in., elongation 5 per cent.

SULPHURIC ACID AND MAGNESIA.

24. The cement shall not contain more than 1.75 per cent. of anhydrous sulphuric acid (SO_3), nor more than 4 per cent. of magnesia (MgO).

2.

1. ABSTRACT EMBODYING PORTIONS APPLICABLE TO CONCRETE REINFORCEMENT FROM THE STANDARD SPECIFICATIONS FOR STRUCTURAL STEEL.

ADOPTED MARCH, 1906, by the
AMERICAN RAILWAY ENGINEERING AND MAINTENANCE OF WAY ASSOCIATION.

Process of Manufacture.

83. Steel shall be made by the open-hearth process.

Schedule of Requirements.

84. The chemical and physical properties shall conform to the following limits:

Elements Considered.		Structural Steel.	Rivet Steel.
Phosphorus, max....	Basic	0.04 per cent.	0.04 per cent.
	Acid	0.08 "	0.04 "
Sulphur, maximum		0.05 "	0.04 "
Ultimate tensile strength.		Desired	Desired
Pounds per square inch		60 000	50 000
Elongation, minimum, percentage in 8 in.		1 500 000*	1 500 000
Fig. 1		Ult. tensile strength	Ult. tensile strength
Character of fracture		Silky	Silky
Cold bends without fracture		180° flat†	180° flat‡

* See paragraph 11. † See paragraphs 12, 13 and 14. ‡ See paragraph 15.

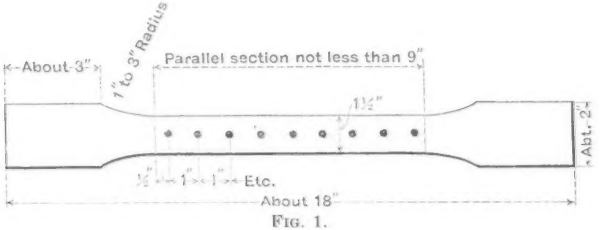


FIG. 1.

The yield point, as indicated by the drop of beam, shall be recorded in the test reports.

Allowable Variations.

85. If the ultimate strength varies more than 4 000 lb. from that desired, a retest shall be made on the same gauge, which, to be acceptable, shall be within 5 000 lb. of the desired ultimate.

Chemical Analyses.

86. Chemical determinations of the percentages of carbon, phosphorus, sulphur, and manganese shall be made by the manufacturer

from a test ingot taken at the time of the pouring of each melt of steel, and a correct copy of such analysis shall be furnished to the engineer or his inspector. Check analyses shall be made from finished material, if called for by the purchaser, in which case an excess of 25% above the required limits will be allowed.

Number of Tests.

92. At least one tensile and one bending test shall be made from each melt of steel as rolled.

Thick Material.

95. Full-sized material for steel 1 in. thick and over, tested as rolled, shall bend, cold, 180° around a pin, the diameter of which is equal to twice the thickness of the bar, without fracture on the outside of bend.

Finish.

98. Finished material shall be free from injurious seams, flaws, cracks, defective edges or other defects, and have a smooth, uniform and workmanlike finish. Plates 36 in. in width and under shall have rolled edges.

Stamping.

99. Every finished piece of steel shall have the melt number and the name of the manufacturer stamped or rolled upon it.

Defective Material.

100. Material which, subsequent to the above tests at the mills, and its acceptance there, develops weak spots, brittleness, cracks, or other imperfections, or is found to have injurious defects, will be rejected at the shop and shall be replaced by the manufacturer at his own cost.

3.

Alternative Specifications for Metal Reinforcement.

A.

1. For wire and rod reinforcements, the steel may be manufactured from Bessemer billets (not re-rolled rails) and shall meet the following requirements:

2. *Tensile Tests.*

Tensile strength, in pounds per square inch, not less than 105 000.

Yield point, in pounds per square inch, not less than 52 500 nor more than 90% of the tensile strength.

Elongation in 8 in., not less than 10%, with the following modifications:

a. For each increase in diameter of $\frac{1}{8}$ in. above $\frac{3}{4}$ in. a deduction of 1% shall be made from the specified elongation.

b. For materials from $\frac{1}{4}$ in. to, but not including, $\frac{5}{16}$ in. diameter the elongation shall be 8 per cent.

For material $\frac{3}{16}$ in. to $\frac{1}{4}$ in., elongation 7 per cent.

For material $\frac{1}{8}$ in. to $\frac{3}{16}$ in., elongation 6 per cent.

For material less than $\frac{1}{8}$ in., elongation 5 per cent.

3. *Bending Tests.*

Test specimens for bending shall be bent, cold, under the following conditions, without fracture on the outside of the bent portion:

Around twice their own diameter:

1 in. diameter, 80 degrees.
$\frac{3}{4}$ in. diameter, 90 degrees.
$\frac{1}{2}$ in. diameter, 110 degrees.

Around their own diameter:

$\frac{1}{4}$ in. diameter, 130 degrees.
$\frac{3}{16}$ in. diameter, 140 degrees.
$\frac{1}{8}$ in. diameter or less, 180 degrees.

B.

Steel wire used for reinforcement should be drawn from bars of basic open-hearth steel of the same quality as that specified for rivet steel.

Test pieces of wire shall bend 180° around their own diameter without fracture.

SUGGESTED FORMULAS FOR REINFORCED CONCRETE CONSTRUCTION.

These formulas are based upon the assumptions and principles given in the chapter on Design.

a. Standard Notation.

1. *Rectangular Beams.*

The following notation is recommended:

f_s = tensile unit stress in steel.
f_c = compressive unit stress in concrete.
E_s = modulus of elasticity of steel.
E_c = modulus of elasticity of concrete.
$n = E_s \div E_c$.
M = moment of resistance, or bending moment in general.
A = steel area.
b = breadth of beam.
d = depth of beam to center of steel.
k = ratio of depth of neutral axis to effective depth, d.
z = depth of resultant compression below top.
j = ratio of lever arm of resisting couple to depth, d.
$jd = d - z$ = arm of resisting couple.
p = steel ratio (not percentage).

I-Beams.

b = width of flange.
b' = width of stem.
t = thickness of flange.

Beams Reinforced for Compression.

A' = area of compressive steel.
p' = steel ratio for compressive steel.

f_s' = unit compressive stress in steel.
C = total compressive stress in concrete.
C' = total compressive stress in steel.
d' = depth to center of compressive steel.
z = depth to resultant of C and C'.

Shear and Bond.

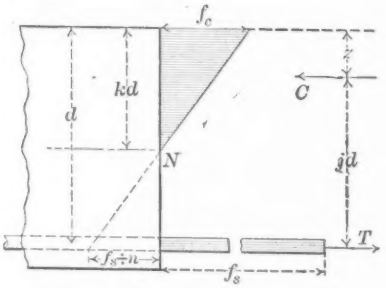
V = total shear.
v = shearing unit stress.
u = bond stress per unit area of bar.
o = circumference or perimeter of bar.
Σ_o sum of the perimeters of all bars.

Columns.

A = total net area.
A_s = area of longitudinal steel.
A_c = area of concrete.
P = total safe load.

b. Formulas.

1. *Rectangular Beams.*



Position of neutral axis,

$$k = \sqrt{2\,p\,n + (p\,n)^2} - p\,n.$$

Arm of resisting couple,

$$j = 1 - \frac{1}{3}k.$$

[For $f_s = 15\,000$ to $16\,000$ and $f_c = 600$ to 650, j may be taken at $\frac{7}{8}$.]

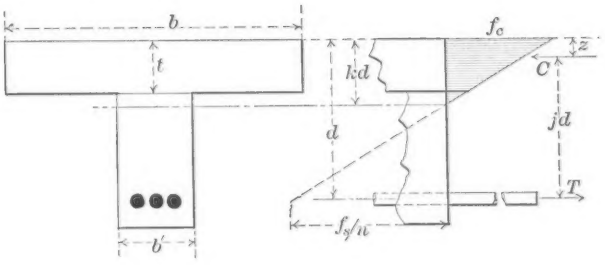
Fiber stresses,

$$f_s = \frac{M}{A\,j\,d} = \frac{M}{p\,j\,b\,d^2}; \quad f_c = \frac{2\,M}{j\,k\,b\,d^2} = \frac{2\,p\,f_s}{k}.$$

Steel ratio,

$$p = \frac{1}{2}\,\frac{1}{\frac{f_s}{f_c}\left(\frac{f_s}{n\,f_c} + 1\right)}.$$

2. T-Beams.



Case I. When the neutral axis lies in the flange, use the formulas for rectangular beams.

Case II. When the neutral axis lies in the stem.

The following formulas neglect the compression in the stem:

Position of neutral axis,

$$k\,d = \frac{2\,n\,d\,A + b\,t^2}{2\,n\,A + 2\,b\,t}.$$

Position of resultant compression,

$$z = \frac{3\,k\,d - 2\,t}{2\,k\,d - t} \cdot \frac{t}{3}.$$

Arm of resisting couple,

$$j\,d = d - z.$$

Fiber stresses,

$$f_s = \frac{M}{A\,j\,d}. \qquad f_c = \frac{M\,k\,d}{b\,t\,(k\,d - \frac{1}{2}\,t)\,j\,d} = \frac{f_s}{n} \cdot \frac{k}{k - 1}.$$

(For approximate results, the formulas for rectangular beams may be used.)

The following formulas take into account the compression in the stem; they are recommended where the flange is small compared with the stem.

Position of neutral axis,

$$k\,d = \sqrt{\frac{2\,n\,d\,A + (b - b')\,t^2}{b'} + \left(\frac{n\,A + (b - b')\,t}{b'}\right)^2} - \frac{n\,A + (b - b')\,t}{b'}.$$

Position of resultant compression,

$$z = \frac{(k\,d\,t^2 - \frac{2}{3}\,t^3)\,b + [(k\,d - t)^2\,(t + \frac{1}{3})(k\,d - t)]\,b'}{t\,(2\,k\,d - t)\,b + (k\,d - t)^2\,b'}.$$

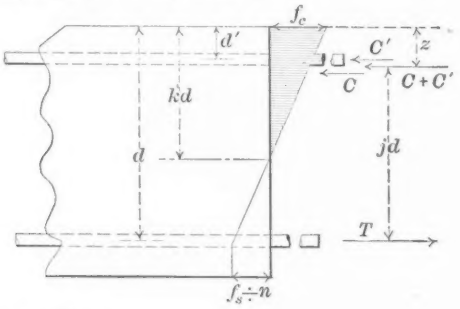
Arm of resisting couple,

$$j\,d = d - z.$$

Fiber stresses,

$$f_s = \frac{M}{A\,j\,d}. \qquad f_c = \frac{2\,M\,k\,d}{[(2\,k\,d - t)\,b\,t + (k\,d - t)^2\,b']\,j\,d}.$$

3. *Beams reinforced for compression,*



Position of neutral axis,

$$k = \sqrt{2\,n\left(p + p'\frac{d'}{d}\right) + n^2\,(p+p')^2} - n\,(p+p').$$

Position of resultant compression,

$$z = \frac{\frac{1}{3}\,k^3\,d + 2\,p'\,n\,d'\left(k - \frac{d'}{d}\right)}{k^2 + 2\,p'\,n\left(k - \frac{d'}{d}\right)}.$$

Arm of resisting couple,

$$j\,d = d - z.$$

Fiber stresses,

$$f_c = \frac{6\,M\,k}{b\,d^2\left[3\,k - k^2 + \frac{6\,p'\,n}{k}\left(k - \frac{d'}{d}\right)\left(1 - \frac{d'}{d}\right)'\right]}$$

$$f_s = \frac{M}{p\,j\,b\,d^2} = n f \frac{1-k}{k}.$$

$$f_s' = n f_c \frac{k - \frac{d'}{d}}{k}.$$

4. *Shear, Bond, and Web Reinforcement.*

In the following formula, Σ_o, refers only to the bars constituting the tension reinforcement at the section in question and $j\,d$ is the lever arm of the resisting couple at the section.

For rectangular beams,

$$v = \frac{V}{b\,j\,d}.$$

$$u = \frac{V}{f\,d\,.\,\Sigma_o}.$$

[For approximate results, j may be taken at $\frac{7}{8}$.]

The stresses in web reinforcement may be estimated by using the following formulas:

Vertical reinforcement,

$$P = \frac{V\,s}{j\,d}.$$

Reinforcement inclined at 75°,

$$P = 0.7\,\frac{V\,s}{j\,d},$$

in which P = stress in single reinforcing member, V = proportion of total shear assumed as carried by the reinforcement, and s = horizontal spacing of the reinforcing members.

The same formulas apply to beams reinforced for compression as regards shear and bond stress for tensile steel.

For T-beams,

$$v = \frac{V}{b'\,j\,d'},\ u = \frac{V}{j\,d\,.\,\Sigma_o}.$$

[For approximate results, j may be taken at $\frac{7}{8}$.]

5. Columns.

Total safe load,

$$P = f_c\,(A_c + n\,A_s) = f_c\,A\,(1 + (n-1)\,p).$$

Unit stresses, $f_c = \dfrac{P}{A\,(1 + (n-1)\,p)}$

$$f_s = n\,f_c.$$

MINORITY REPORT.

To the Members of the American Society of Civil Engineers:

The undersigned, members of the Special Committee on Concrete and Reinforced Concrete, who did not indorse the majority report of the Committee as submitted to the American Society of Civil Engineers, beg leave to present their reasons for dissenting in some very essential points from the views of the majority of the members of the Committee.

1. Form of Report.—We do not consider the report as presented to be in proper shape for a document to be submitted to a society composed of engineers. As we understand the duty of your Committee to be "to investigate and report", we are of the opinion that the report should consist of a dissertation on our investigations, with the conclusions derived therefrom. Some chapters of this report are more in the shape of specifications or arbitrary rules, such as an engineer might prepare for the use of his subordinates. We consider the report as a whole to be hastily prepared, and neither carefully considered, nor systematically arranged.

2. Design.—The chapter on design is limited to a description of minor details of a particular kind of construction, generally, and sometimes improperly, used in reinforced concrete work of buildings, to the exclusion of other types; to which limitations a designer with more advanced ideas, who does not follow traditions, would naturally object. As the art of reinforced concrete construction is yet in its infancy, the tentative designs and methods adopted by the pioneers in their attempts to utilize a new material cannot be expected to be perfect, but more or less crude and unscientific, as has been the case in structures built of other materials, which, by the natural process of evolution, have gradually developed into more rational forms.

Had the report been prepared at the time your Committee was appointed, about four years ago, when this so-called progress report could have been made just as well as now, it would probably have been considered as giving a fair résumé of the state of the art at that time. The progress made and the experience gained in the last four years have not been given sufficient attention, and we consider the recommendations referring to design not broad enough and not up to date.

3. Working Stresses.—Scientific facts are not established by a vote. They should be deduced from experimental data and practical experience, and should appeal to reason and judgment. The working strains on concrete, as recommended, are arbitrarily assumed, without reference to any tests, or facts established by experience, and without giving any reason for departing from the practice established by years

of experience. In other words, the members of the Society are asked to believe in the superior judgment of the members of the Committee who are responsible for this report.

The properties of reinforced concrete are not yet as well established as those of nearly all other building materials, owing to the lack of conclusive tests. No recognized theory for computing the internal forces in reinforced concrete is in existence at the present time, and as the data which are now available have not yet been sufficiently confirmed by practical experience, they should be used with the utmost caution, so that if there be an error, it will be on the side of safety. The number of disasters in concrete structures has not diminished, but rather increased; a word of warning, therefore, would be appropriate at this time.

We are of the opinion that the acceptance of the report in its present shape would have a tendency to encourage questionable practice, which is not in harmony with the aims and objects of this Society.

It may be said that the Society disclaims all responsibility for statements and opinions expressed in any of its publications; nevertheless, the statements embodied in a report of a Special Committee appointed to report on a special subject by the American Society of Civil Engineers, if printed in its *Transactions,* will be quoted as being indorsed by that Society.

The members forming the minority of the Special Committee on Concrete and Reinforced Concrete are of the opinion that the majority report should be referred back to the Committee for revision.

Respectfully submitted,

JANUARY, 1909.

C. C. SCHNEIDER, *Chairman,*
EMIL SWENSSON,
J. E. GREINER,
OLAF HOFF.

EXCURSIONS AND ENTERTAINMENTS AT THE FIFTY-SIXTH ANNUAL MEETING.

Wednesday, January 20th, 1909.—After the Business Meeting, lunch for over 700 members was served at 1 P. M., at the Society House.

At 3 P. M., by invitation of the Commissioner of the Department of Bridges and of the Commissioner of the Department of Docks and Ferries, there was an excursion to the Queensboro (Blackwell's Island) Bridge and the Williamsburg Bridge, and then to the Chelsea Section of Docks, North River. The party was conveyed in automobiles furnished through the courtesy of the Pennsylvania Steel Company and of Messrs. Snare and Triest and Messrs. R. P. and J. H. Staats. Those who could not be accommodated in the automobiles were taken by steamer, provided through the courtesy of the Commissioner of Charities, from the pier at the foot of East Sixtieth Street, down the East River, around the Battery and up the North River to the Chelsea Piers, where refreshments were served.

At 9 P. M., there was a reception, with dancing, in the Society House, at which the attendance was about 350.

Thursday, January 21st, 1909.—The day was devoted to an excursion to the Ashokan Reservoir by invitation of the Board of Water Supply. The party took the ferry at the foot of West Forty-second Street at 9 A. M., and the journey to Brown's Station was made by special train over the West Shore, and Ulster and Delaware Railroads, stops being made at Newburgh and Kingston.

The excursion afforded an excellent opportunity to view a number of historic points on the Hudson, and to inspect the basin of the Ashokan Reservoir and Olive Bridge Dam and the sites of the several dams and dikes, as well as the contractors' plant.

On the arrival of the party lunch was served at the contractors' camp by invitation of Messrs. MacArthur Bros. Co. and Winston and Co.

The train returning left Olive Bridge Dam at 3 P. M. and reached New York City at 6.30 P. M.

In the evening, at the Society House, there was a social and informal "Smoker," at which there was an attendance of more than 700.

The following list contains the names of 726 members, of various grades, who registered as being in attendance at the Annual Meeting. The list is incomplete, as some members failed to register, and it does not contain the names of any of the guests of the Society or of individual members. The number of guests is estimated at 400.

Adams, Arthur...New York City
Aiken, W. A....Philadelphia, Pa.
Aims, Walton I...New York City
Alderson, A. B., West Hartford, Conn.
Alexander, H. J..New York City

Allardice, E. R. B..Clinton, Mass.
Allen, C. Frank....Boston, Mass.
Allen, Henry C...Syracuse, N. Y.
Allen, Kenneth...New York City
Andrews, Horace...Albany, N. Y.
Archbald, James....Scranton, Pa.
Armstrong, R. S..New York City
Ash, Dorsey..San Francisco, Cal.
Atkinson, A.New Brunswick, N. J.
Atwood, T. C.....Yonkers, N. Y.
Auryansen, F.....Jamaica, N. Y.

Babcock, W. S....New York City
Baird, H. C.....New York City
Baldwin, F. H....Brooklyn, N. Y.
Ball, C. B.....Chicago, Ill.
Bance, C. W....Jersey City, N. J.
Barker, J. M.....Boston, Mass.
Barnes, M. G.....Albany, N. Y.
Barnes, T. H.,
Puerto Barrios, Guatemala
Barney, P. C.....Brooklyn, N. Y.
Barrett, R. E.....New York City
Barrows, H. K.....Boston, Mass.
Barshell, F. B....New York City
Bascom, H. F.....Allentown, Pa.
Basinger, J. G....New York City
Bates, Onward.....Chicago, Ill.
Baucus, W. I.....Havana, Cuba
Baum, George....Yonkers, N. Y.
Becker, E. J....Waterford, N. Y.
Beer, Paul.....Des Moines, Iowa
Belden, E. T.....Pittsfield, Mass.
Belknap, J. M....New York City
Belknap, W. E....New York City
Bellinger, L. F..Bath Beach, N. Y.
Belzner, T.....New York City
Bensel, J. A.....New York City
Benton, L. S.....New York City
Benzenberg, G. H.,
Milwaukee, Wis.
Berger, Bernt....New York City
Bernstein, L.....Baltimore, Md.
Bettes, C. R..Far Rockaway, N. Y.
Betts, R. T.....Brooklyn, N. Y.

Beugler, E. J.....New York City
Bieler, A. H....Englewood, N. J.
Bissell, H.....W. Medford, Mass.
Blakeslee, C...New Haven, Conn.
Blanchard, A. H.,
Providence, R. I.
Blanchard, M.....New York City
Bland, J. C.....Pittsburg, Pa.
Blythe, L. H....Philadelphia, Pa.
Boardman, H. E..New York City
Bogert, C. L.....New York City
Boller, A. P...East Orange, N. J.
Boller, A. P., Jr...New York City
Bond, G. M.....Hartford, Conn.
Boniface, A.....Scarsdale, N. Y.
Booth, G. W.....New York City
Boucher, W. J....New York City
Bower, C. P....Philadelphia, Pa.
Bowman, A. L....New York City
Boyd, J. C.....Montclair, N. J.
Boyd, R. W.....New York City
Brackenridge, J. C.,
Richmond Hill, N. Y.
Brackenridge, W. A.,
Buffalo, N. Y.
Brackett, Dexter....Boston, Mass.
Bradley, F. E.....New York City
Branne, J. S..Mt. Vernon, N. Y.
Braune, G. M.....Albany, N. Y.
Breckinridge, W. L..Chicago, Ill.
Breed, C. B.....Boston, Mass.
Brendlinger, P. F.,
Philadelphia, Pa.
Brodie, O. L.....New York City
Brown, C. E.,
Hudson Heights, N. J
Brown, W. L.....Montclair, N. J.
Brown, W. P.....New York City
Burden, James.....Troy, N. Y.
Burgess, G. H....New York City
Bush, E. W.....Hartford, Conn.
Bush, L.....East Orange, N. J.

Caldwell, G. B....Yonkers, N. Y.
Cantine, E. I..East Orange, N. J.

Cantwell, H. E....New York City
Carpenter, A. W..Yonkers, N. Y.
Carpenter, C. E...Yonkers, N. Y.
Carr, Albert...East Orange, N. J.
Chadwick, C. R...New York City
Chambers, R. H...New York City
Chappell, T. F....New York City
Chase, J. C..Derry Village, N. H.
Chase, R. D..New Bedford, Mass.
Chase, W. H.....Flushing, N. Y.
Chester, J. N.....Pittsburg, Pa.
Christian, G. L...New York City
Christie, J.....Philadelphia, Pa.
Christy, G. L.....New York City
Church, E. C.....New York City
Churchill, C. S.....Roanoke, Va.
Clapp, O. F.....Providence, R. I.
Clark, G. H.....New York City
Clarke, E. W.....Babylon, N. Y.
Clarke, G. C.....New York City
Clarke, St. John....Bogota, N. J.
Clement, F. H...Philadelphia, Pa.
Closson, E. S.....Brooklyn, N. Y.
Coburn, H. B., Jr.New York City
Codwise, E. B....Kingston, N. Y.
Cohen, F. W.....Harrisburg, Pa.
Cole, G. N.....New York City
Cole, H. J.....East Orange, N. J.
Cole, H. N.....Providence, R. I.
Collier, B. C.....New York City
Collingwood, F...Elizabeth, N. J.
Conger, A. A.....New York City
Conkling, C. C.....Buffalo, N. Y.
Cook, F. S.....Yonkers, N. Y.
Cook, J. H.....Passaic, N. J.
Cook, J. W.....Passaic, N. J.
Coombs, S. E.....Yonkers, N. Y.
Coomer, R. M....New York City
Cooper, S. L.....Yonkers, N. Y.
Cornell, J. N. H..New York City
Coyne, H. L.....New York City
Crandall, C. L.....Ithaca, N. Y.
Crane, A. S.....New York City
Crane, F. E....Amsterdam, N. Y.
Crane, J. S.....Newark, N. J.
Crane, W. E.....Brooklyn, N. Y.
Cresson, B. F., Jr.New York City
Creuzbaur, R. W..Brooklyn, N. Y.
Crooks, C. H.....New York City
Crowell, Foster...New York City
Cuddeback, A. W..Paterson, N. J.
Cullen, J. F....Philadelphia, Pa.
Curtis, F. S.....Boston, Mass.

Daggett, F. W.....Boston, Mass.
Davis, A. L....Mt. Vernon, N. Y.
Davis, A. P....Washington, D. C.
Davis, C. E..Brown Station, N. Y.
Davis, J. L.....New York City
Deans, J. S.....Phœnixville, Pa.
de Gratresse, J. R.New York City
Dennis, W. F..Washington, D. C.
Develin, R. G....Philadelphia, Pa.
Devin, George....New York City
Devlin, H. S....Brooklyn, N. Y.
de Wyrall, C.,
Ridgefield Park, N. J.
Deyo, S. L. F.....New York City
Dimon, D. Y.....Passaic, N. J.
Dodge, S. D.,
Cornwall-on-Hudson, N. Y.
Donham, B. C....New York City
Dorrance, W. T...New York City
Dougherty, R. E..New York City
Douglas, W. J..Washington, D. C.
Dunham, H. F....New York City

Earle, Thomas.....Steelton, Pa.
Easterbrooks, P. B.,
Brooklyn, N. Y.
Eddy, H. P.....Boston, Mass.
Edwards, J. H.....Passaic, N. J.
Ehrbar, L. H....New York City
Ellis, J. W.....Woonsocket, R. I.
Ely, C. B.....Harrisburg, Pa.
Emerson, Guy C....Boston, Mass.
Emmons, C. M..Beaver Falls, Pa.
Engle, R. L..Johnson City, Tenn.
Erlandsen, O.....New York City
Esselstyn, H. H..Brooklyn, N. Y.

Evans, E. G....Hampton, Canada
Evans, J. M.....Brooklyn, N. Y.
Everham, A. C.....Detroit, Mich.

Falk, M. S.....New York City
Farley, J. M..White Plains, N. Y.
Farnham, R., Jr.,
Washington, D. C.
Fay, F. H.....Boston, Mass.
Fehr, H. R.....Easton, Pa.
Felton, H. C.....Camden, N. J.
Fenton, L. G.....Brooklyn, N. Y.
Ferguson, G. R...New York City
Ferris, F. E.....Jersey City, N. J.
Fetherston, J. T.,
New Brighton, N. Y.
Firth, E. W.....Jamaica, N. Y.
Fisher, E. A.....Rochester, N. Y.
Fisher, F. D.....Brooklyn, N. Y.
Fisher, H. T....Philadelphia, Pa.
Fitch, C. L.....Brooklyn, N. Y.
Fletcher, Robert..Hanover, N. H.
Flinn, A. D.....New York City
Ford, Harry C....New York City
Ford, W. G.....Brooklyn, N. Y.
Fougner, H.....New York City
Francis, G. B....New York City
Fraser, C. E.,
Richmond Hill, N. Y.
Frazee, J. H.....New York City
Frazer, J. S.....Brooklyn, N. Y.
French, A. H....Brookline, Mass.
French, A. W...Worcester, Mass.
French, J. B.....New York City
French, M. J.....Utica, N. Y.
Frost, G. H.....New York City
Frost, G. S.....Brooklyn, N. Y.
Frye, A. I.....New York City
Fuller, F. L.....Boston, Mass.
Fuller, G. W.....New York City
Fuller, W. B.....New York City
Fuller, W. E.....Brooklyn, N. Y.
Furber, W. C...Philadelphia, Pa.

Gahagan, W. H..Brooklyn, N. Y.
Gaiger, F. M.....New York City
Gandolfo, J. H....New York City
Gardiner, F. W....Yonkers, N. Y.
Gartensteig, C....New York City
Gaston, L. P.....Somerville, N. J.
Gerber, E.....Pittsburg, Pa.
Gifford, G. E.....New York City
Gildersleeve, A. C..New York City
Giles, Robert.....New York City
Gillen, W. J.Brown Station, N. Y.
Gillespie, R. H...New York City
Goad, C. E.....Toronto, Canada
Goddard, H. W.....Boston, Mass.
Goodnough, X. H....Boston, Mass.
Goodrich, E. P...New York City
Goodridge, J. W..Brooklyn, N. Y.
Goodsell, D. B....New York City
Gowen, C. S.....Ossining, N. Y.
Grady, C. B..West Orange, N. J.
Granbery, J. H....New York City
Gray, J. H.....Orange, N. J.
Gray, William....New York City
Green, B. R....Washington, D. C.
Green, C. N.....New York City
Green, F. M.....New York City
Greene, Carleton..New York City
Greene, G. S., Jr..New York City
Gregory, C. E....Mt. Kisco, N. Y.
Greiner, J. E.....Baltimore, Md.
Grimes, E. L.....Troy, N. Y.
Grimm, C. R....Brooklyn, N. Y.
Gross, C. A....So. Bethlehem, Pa.
Grossart, L. J. H..Allentown, Pa.
Grover, N. C....New York City
Gustin, R. P...Mt. Vernon, N. Y.
Guthrie, K. O....Brooklyn, N. Y.

Hague, C. A.....New York City
Haight, S. S.....New York City
Haines, C. W....Philadelphia, Pa.
Haines, E. G.,
Richmond Hill, N. Y.
Hale, H. M.....New York City
Hall, M. W.....New York City
Hallock, J. C.....Newark, N. J.
Hancock, R. R....New York City

Hansel, Charles...New York City
Haring, A.....New York City
Harlow, G. R.....Pittsburg, Pa.
Harrington, F. F....Norfolk, Va.
Harrison, E. W..Jersey City, N. J.
Harte, C. R...West Haven, Conn.
Harwi, S. J.....Bayonne, N. J.
Haskins, W. J....New York City
Hauck, W.....Brewster, N. Y.
Haviland, A.....New York City
Hawley, C. R.....Buffalo, N. Y.
Hayes, H. W.....Boston, Mass.
Hayes, S. W.....Geneva, N. Y.
Hazard, E.....Mauch Chunk, Pa.
Hazard, S....White Plains, N. Y.
Hazelton, C. W.,
Turners Falls, Mass.
Hazen, Allen.....New York City
Hazen, W. N.....New York City
Healy, J. R.....New York City
Hedden, E. B.,
White Plains, N. Y.
Heller, J. W..West Orange, N. J.
Hench, N. M.....Pittsburg, Pa.
Henderson, J. T..Hartford, Conn.
Herbert, H. M.Bound Brook, N. J.
Hering, R.....Montclair, N. J.
Herrick, J. J....Flushing, N. Y.
Herschel, C.....New York City
Hewes, V. H.....New York City
Higgins, C. H..Jersey City, N. J.
Higgins, J. W.,
Roselle Park, N. J.
Higginson, J. Y.,
New Rochelle, N. Y.
Hildreth, J. L., Jr.,
Babylon, N. Y.
Hill, W. R.....Albany, N. Y.
Hoag, S. W., Jr..New York City
Hodgdon, F. W.....Boston, Mass.
Hoff, Olaf.....Tarrytown, N. Y.
Hogan, J. P....High Falls, N. Y.
Holbrook, P.....New York City
Holden, C. A.....Hanover, N. H.
Holmes, G. D....Syracuse, N. Y.
Holtsmark, E....Brooklyn, N. Y.
Honness, G. G.,
Pleasantville, N. Y.
Hood, J. N..Richmond Hill, N. Y.
Horan, J. J.....Brooklyn, N. Y.
Horne, Harold W.,
Cornwall-on-Hudson, N. Y.
Horton, H. E.....Chicago, Ill.
Hovey, O. E.....Plainfield, N. J.
Howard, J. L.....Melrose, Mass.
Howell, W. A.....Newark, N. J.
Hoyt, J. C.....Washington, D. C.
Hubbell, G. S....Flushing, N. Y.
Hudson, C. W....New York City
Huestis, C. C.....Buffalo, N. Y.
Hughes, H. J...Cambridge, Mass.
Hulse, S. C.....Bedford, Pa.
Humphrey, R. L.,
Philadelphia, Pa.
Hunt, C. E.....New York City
Hunt, Charles Warren,
New York City
Huntington, G. D..Massena, N. Y.
Hurst, G. J.....New York City

Ingersoll, C. M., Jr.,
New York City
Irwin, J. C.....New York City
Ives, A. S....Poughkeepsie, N. Y.

Janes, G. P.....Roselle, N. J.
Japp, Henry....Richmond, N. Y.
Jenkins, J. E.....Medina, N. Y.
Jewel, L. L.....Pittsburg, Pa.
Johannesson, S...New York City
Johnson, L. J...Cambridge, Mass.
Johnson, T. H.....Pittsburg, Pa.
Johnson, W. S.....Boston, Mass.
Jones, Henry L...New York City
Jones, I. P.....Georgetown, Del.
Jonson, E. F.....New York City
Jordan, E. C.....Portland, Me.
Jouett, H. D.....New York City
Judson, W. P.....Oswego, N. Y.
Just, G. A.....Scarsdale, N. Y.

Kastl, A. E.....Peekskill, N. Y.
Keith, H. C.....New York City
Keller, O. B.....New York City
Kelley, W. D.....Yonkers, N. Y.
Khuen, R.....Pittsburg, Pa.
Killam, C. W...Cambridge, Mass.
Kimball, G. A.....Boston, Mass.
King, C. C..New Brighton, N. Y.
King, H. W.....Cleveland, Ohio
King, P. S.....Philadelphia, Pa.
King, W., Jr.....New York City
Kirkwood, H. C..New York City
Klapp, Eugene....Wyckoff, N. J.
Knap, E. D.....Yonkers, N. Y.
Knap, Joseph M..New York City
Knickerbocker, C. E.,
Middletown, N. Y.
Knighton, J. A...New York City
Kolb, H. J.....Brooklyn, N. Y.
Kornfeld, A. E...New York City
Kroeber, A. T.....Reading, Pa.

La Chicotte, H. A.New York City
Landers, C. S....New York City
Langthorn, J. S..Kingston, N. Y.
Larsson, C. G. E..Plainfield, N. J.
Lathrop, J. C..Jersey City, N. J.
Lavis, F.....Mt. Vernon, N. Y.
Lea, R. S.....Montreal, Canada
Leather, B. H....New York City
Lee, W. B.....Hillburn, N. Y.
Leffingwell, F. D.,
Upper Montclair, N. J.
Lentilhon, E....Bay Shore, N. Y.
Leonard, C. M.....Chicago, Ill.
Leonard, H. R..Philadelphia, Pa.
Lesley, R. W...Philadelphia, Pa.
Lewis, E. W...New Haven, Conn.
Lewis, N. P.....New York City
Lex, W. I.....Philadelphia, Pa.
Lindenthal, G.....New York City
Llewellyn, F. J.....Chicago, Ill.
Llewellyn, F. T...New York City
Lobo, C.....Brooklyn, N. Y.
Lockwood, J. A...New York City
Lockwood, W. D.,
Schenectady, N. Y.
Logan, W. S....Arlington, N. J.
Long, E. McL....New York City
Look, M. J..Brown Station, N. Y.
Loomis, H.....Mt. Vernon, N. Y.
Low, G. E.....New York City
Loweth, C. F.....Chicago, Ill.
Lundie, J.....New York City
Lundoff, C. W....Cleveland, Ohio
Luster, W. H., Jr.,
Elizabeth, N. J.
Lynn, H. H. E....New York City

MacCracken, G. G., New York City
Macdonald, C., Gananoque, Canada
MacGregor, R. A..New York City
Machen, H. B....New York City
Macksey, H. V.....Boston, Mass.
McBurney, H.West Haven, Conn.
McCarthy, G. A.,
North Bay, Canada
McClave, S. W., Jr.,
Grantwood, N. J.
McCord, J. B.....Elizabeth, N. J.
McCurdy, H. S. R.,
Brown Station, N. Y.
McInnes, F. A.....Boston, Mass.
McIntire, T. B....Yonkers, N. Y.
McKenzie, T. H.,
Southington, Conn.
McLure, N. R....Phœnixville, Pa.
McMinn, T. J.....New York City
McNaugher, D. W..Pittsburg, Pa.
McNeal, J.....Easton, Pa.
McPherson, R. H..New York City
Madden, J. H.....New York City
Maier, H. L.....Wilmington, Del.
Maignen, J. P. A.,
Philadelphia, Pa.
Mair, J. W....Mt. Vernon, N. Y.
Manley, Henry.....Boston, Mass.
Mansfield, R. J...New York City
Marburg, E.....Philadelphia, Pa.
Marden, H. H., Jr.New York City

Marden, W. R.....Albany, N. Y.
Marple, W. McK..Scranton, Pa.
Matheson, J. D.Winnipeg, Canada
Mayell, A. J.....New York City
Mead, C. A.,
Upper Montclair, N. J.
Meadowcroft, W..New York City
Mebus, C. F....Philadelphia, Pa.
Meem, J. C.....Brooklyn, N. Y.
Meggy, R. L. G..Plainfield, N. J.
Mehren, E. J.....New York City
Melius, L. L.....New York City
Merrill, Ogden....New York City
Merriman, T.....New York City
Merritt, D. S....Tarrytown, N. Y.
Merryman, W. C..New York City
Metcalf, Leonard...Boston, Mass.
Meyers, C. W.....New York City
Miller, Frank....New York City
Miller, H. C.....New York City
Miller, H. A.,
Newton Highlands, Mass.
Miller, R. P.....New York City
Mills, C. M.....Philadelphia, Pa.
Minor, E. E...New Haven, Conn.
Mitchell, S. P..Philadelphia, Pa.
Modjeski, Ralph.....Chicago, Ill.
Moisseiff, L. S....New York City
Molitor, David A....Ithaca, N. Y.
Molitor, F.....Philadelphia, Pa.
Monahan, J. J.,
West Chelmsford, Mass.
Moncure, W. A..Philadelphia, Pa.
Montfort, R.....Louisville, Ky.
Moore, W. H..New Haven, Conn.
Morse, G. F.....Bayonne, N. J.
Morton, W. S....New York City
Moulton, Guy....Syracuse, N. Y.
Moulton, Mace...New York City
Musson, E. F.....Norwich, N. Y.
Myers, C. H.....New York City
Myers, J. H..White Plains, N. Y.

Neale, John C.....Pittsburg, Pa.
Nelson, J. C.....New York City
Nethercut, E. S.....Chicago, Ill.
Neumeyer, R. E..Bethlehem, Pa.
Nial, W. A.....New York City
Nichol, H. S.....New York City.
Nichols, A. R..Washington, D. C.
Nichols, C. H..New Haven, Conn.
Noble, Alfred.....New York City
North, Edward P..New York City
Northrop, A. A.,
Brown Station, N. Y.
Nye, A. S.....New York City

Oakley, G. I...Little Falls, N. Y.
Obreiter, J. W....Hoboken, N. J.
O'Brien, J. H....New York City
Odell, F. S...Port Chester, N. Y.
Ogden, Henry N....Ithaca, N. Y.
Olmsted, A. E....New York City
Opdyke, S. B., Jr..New York City
O'Rourke, J. F....New York City
Orr, Alexander.Gloversville, N. Y.
Orrok, G. A.....Brooklyn, N. Y.
Osborn, F. C.....Cleveland, Ohio
Osgood, J. O...Jersey City, N. J.
Ostrup, J. C.....New York City
Ott, S. J.....Rutherford, N. J.
Owen, James....Montclair, N. J.
Owen, K. D.....Montclair, N. J.
Oxholm, T. S.,
West New Brighton, N. Y.

Parker, C. J.... .New York City
Parker, William....Boston, Mass.
Parks, O. E.....Westfield, Mass.
Parmley, W. C.,
Upper Montclair, N. J.
Parsons, G. W.....Steelton, Pa.
Parsons, H. A...Stamford, Conn.
Parsons, H. de B..New York City
Parsons, W. B....New York City
Pegram, G. H....New York City
Pendlebury, E....Arlington, N. J.
Perkins, P. S....Providence, R. I.
Perry, J. P. H....New York City
Peterson, John....New York City

Pflueger, A. C....New York City
Phelps, W. C.....New York City
Philips, J. H.....Newark, N. J.
Phillips, Asa E. Washington, D. C.
Pike, R. A.....Mt. Vernon, N. Y.
Plogsted, W. J....New York City
Pohl, C. A.....New York City
Polk, W. A.....New York City
Pollock, C. D.....New York City
Poole, C. A.....Waterford, N. Y.
Porter, J. E.....Yonkers, N. Y.
Post, Henry W.,
Richmond Hill, N. Y.
Potter, A.....New York City
Potts, Clyde.....New York City
Pratt, A. H.....New York City
Pratt, W. A.....New York City
Price, P. L....East Orange, N. J.
Priest, B. B...East Orange, N. J.
Pruyn, F. L.....New York City
Pugh, M. R.....Philadelphia, Pa.
Putnam, G. R..Washington, D. C.

Quick, A. M.....Baltimore, Md.
Quincy, C. F.....New York City

Read, R. L.....Malden, Mass.
Reid, H. A...Dobbs Ferry, N. Y.
Rennell, H. H..Bridgeport, Conn.
Reynders, J. V. W...Steelton, Pa.
Rice, G. S.....New York City
Richardson, C....New York City
Richardson, J. D...Corona, N. Y.
Richardson, J. H...Orange, N. J.
Ridgway, R..Poughkeepsie, N. Y.
Riggs, M. J.....Toledo, Ohio
Rights, L. D.....New York City
Ripley, H. L....Providence, R. I.
Ripley, J.....Albany, N. Y.
Robb, L. A.....Newark, N. J.
Robbins, D. W....New York City
Robbins, H. R....New York City
Roberts, P., Jr..Philadelphia, Pa.
Robinson, A. W.,
Montreal, Canada
Robinson, E. W...New York City
Robinson, R. T. Weehawken, N. J.
Rogers, H. L.....Brooklyn, N. Y.
Rohrer, J. B.....Lancaster, Pa.
Rose, R. V..Niagara Falls, N. Y.
Ruggles, W. B..Cambridge, Mass.
Ryan, M. H.....New York City
Ryder, E. M. T...New York City

Sabin, A. H.....Flushing, N. Y
Sanford, H. C....New York City
Sass, C. W.....New York City
Saunders, G. C...New York City
Sayers, E. L.....New York City
Sayles, R. W.....New York City
Schaeffer, Amos...New York City
Schaeffler, J. C.....Boston, Mass.
Schall, F. E.,
South Bethlehem, Pa.
Schmidt, Max E...Convent, N. J.
Schneider, C. C.,
Philadelphia, Pa.
Schwarze, C. T.,
South Orange, N. J.
Schwitzer, J. E.,
Winnipeg, Canada
Scorgie, J. C...Cambridge, Mass.
Seaman, H. B....New York City
Searle, C. D.....New York City
Searle, L. F.,
Brown Station, N. Y.
Senior, F. S...Montgomery, N. Y.
Shaw, D. J.....Chappaqua, N. Y.
Shellenberger, L. R.,
Bayonne, N. J.
Sherrerd, J. M.....Easton, Pa.
Sherrerd, M. R.....Newark, N. J.
Shoemaker, M. N...Newark, N. J.
Simpson, G. F....New York City
Sitt, W. T.....New York City
Skillin, E. S....Glen Ridge, N. J.
Skinner, F. W.,
Tompkinsville, N. Y.
Skinner, J. F....Rochester, N. Y.
Slocum, C. L..New Haven, Conn.

Smith, E. G.....New York City
Smith, F. C.,
Great Barrington, Mass.
Smith, H. S....Wilkes-Barre, Pa.
Smith, J. Waldo..New York City
Smith, M. H..White Plains, N. Y.
Smith, T. G.....Buffalo, N. Y.
Smith, W. M., Sr.,
West New Brighton, N. Y.
Snow, J. B..Richmond Hill, N. Y.
Snyder, C. H.....Oswego, N. Y.
Soper, G. A.....New York City
Souther, T. W..West Point, N. Y.
Sparrow, W. W. K.,
Kansas City, Mo.
Spear, P. H.....Raritan, N. J.
Spear, W. E.....Babylon, N. Y.
Spencer, W. T..New Haven, Conn.
Sperry, H. M.....New York City
Splitstone, C. H.,
East Orange, N. J.
Spofford, C. M....New York City
Spooner, A. W....New York City
Sprague, E. L., Jr.,
Valhalla, N. Y.
Stanford, H. R..Philadelphia, Pa.
Staniford, C. W..Brooklyn, N. Y.
Stanton, R. B.....New York City
Starr, H. H.....Philadelphia, Pa.
Stearns, F. L....Scarsdale, N. Y.
Stearns, F. P.....Boston, Mass.
Stearns, R. H.....New York City
Stern, E. W.....New York City
Stevenson, W. F..New York City
Stewart, W. J....Rochester, N. Y.
Stickney, G. F.....Albany, N. Y.
Stilson, M. A. O.,
Waterbury, Conn.
Stoddard, G. C....New York City
Stoddard, R. F....Milford, Conn.
Stone, E. E.....Boston, Mass
Stone, W. W.....Walden, N. Y.
Storey, W. B., Jr....Chicago, Ill.
Stowell, C. F.....Albany, N. Y.
Stowitts, G. P....New York City
Strobel, C. L.....Chicago, Ill
Strouse, W. F.....Baltimore, Md.
Stuart, Francis Lee,
Essex Fells, N. J.
Suter, R.....New York City
Sutton, F.....Washington, D. C
Swain, G. F.....Boston, Mass.
Swanker, J. E..Schenectady, N. Y.
Swensson, E.....Pittsburg, Pa.
Swift, W. E.,
Cornwall-on-Hudson, N. Y.
Swindells, J. S....Jamaica, N. Y.

Taber, G. A.....Brooklyn, N. Y.
Tait, Harold.....New York City
Tait, J. G.....Metuchen, N. J.
Talbot, A. N.....Urbana, Ill.
Tallman, P. B..East Orange, N. J.
Tarr, C. W.....Babylon, N. Y.
Taylor, A. J....Wilmington, Del.
Taylor, C. F.....New York City
Taylor, E. A.....Uxbridge, Mass.
Taylor, L. A.....Boston, Mass
Taylor, S. A.....Pittsburg, Pa.
Tenney, G. O.....Richmond, Va.
Terry, J. H.....Philadelphia, Pa.
Theban, J. G.....New York City
Thomas, C. D....Brooklyn, N. Y.
Thomes, E. H.....Jamaica, N. Y.
Thompson, F.....Boston, Mass.
Thompson, S. E.,
Newton Highlands, Mass.
Thomson, T. K...New York City
Thornley, J.....Albany, N. Y.
Tillson, G. W....Brooklyn, N. Y.
Tingley, R. H....New York City
Tinkham, S. E.....Boston, Mass.
Titsworth, R. B..Plainfield, N. J.
Tompkins, E. de V..Pelham, N. Y.
Tórrey, J. E.....Paterson, N. J.
Travell, W. B....Plainfield, N. J.
Tretter, G. A.....Roanoke, Va.
Tribus, L. L.....New York City
Triest, W. G.....New York City
Trout, C. E.....New York City

True, A. O.....New York City
Tull, R. W.....New York City
Turner, D. L.....New York City
Turner, E. K.....Boston, Mass.
Turner, H. C.....New York City
Turner, W. T.....Lima, Peru
Tuttle, A. S.....New York City

Van Cleve, A. H.,
Niagara Falls, N. Y.
Van Horne, J. G..New York City
Vanneman, C. R....Albany, N. Y.
Van Norden, E. M.,
Brooklyn, N. Y.
Van Suetendael, A. O.,
Albany, N. Y.
Verrill, G. E..New Haven, Conn.
Vier, H.....White Plains, N. Y
Vogleson, J. A..Philadelphia, Pa.
Volck, A. G.....Glen Cove, N. Y.
Voorhees, P.....Harrisburg, Pa
Vorce, C. B.....New York City

Waddell, J. A. L.,
Kansas City, Mo.
Waddell, N. E.,
Vancouver, Canada
Wadsworth, J. E..New York City
Wagner, B. M....Brooklyn, N. Y.
Wagner, H. E.....Andover, N. J.
Wagner, J. C....Philadelphia, Pa.
Waldron, S. P..East Orange, N. J.
Walker, J. W.....Pittsburg, Pa.
Ward, C. D.....New York City
Warren, H. A....New York City
Wason, L. C.....Brookline, Mass.
Wassner, M.....New York City
Watkins, F. W.,
White Plains, N. Y.
Watt, J. R.....Albany, N. Y.
Webster, A. L.....New York City
Webster, G. S...Philadelphia, Pa.
Wegmann, E.....Yonkers, N. Y
Wells, C. E..White Plains, N. Y.
Wendt, E. F.....Pittsburg, Pa.
West, O. J.....Chicago, Ill.
Weston, R. S.....Boston, Mass.
Wheelock, De F. A...Warren, Pa.
Whinery, S....East Orange, N. J.
Whipple, G. C....New York City
Whiskeman, J. P..New York City
White, T. S.....Beaver Falls, Pa
White, W. M.....New York City
Whittemore, W. F..Hoboken, N. J.
Wiggin, E. W..New Haven, Conn.
Wiggin, T. H.,
White Plains, N. Y
Wilcock, F.....Brooklyn, N. Y.
Wild, H. J.....Weehawken, N. J
Wiley, W. H.....New York City
Wilkes, J. K..New Rochelle, N. Y.
Wilkins, W. G.....Pittsburg, Pa.
Williams, G. S..Ann Arbor, Mich.
Williamson, F. S..New York City
Wilson, C. W. S.,
New Rochelle, N. Y.
Wilson, H. M..Washington, D. C.
Wilson, P. H....Philadelphia, Pa.
Wilson, W. W.....New York City
Winsor, F. E..White Plains, N. Y.
Wise, C. R.....Passaic, N. J.
Wölfel, P. L.....Pittsburg, Pa.
Wolverton, I. M.,
Mt. Vernon, Ohio
Wood, H. B.....Belmont, Mass.
Woodard, S. H...Scarsdale, N. Y.
Woolley, A. F.,
Sylvan Beach, N. Y.
Worcester, J. R.....Boston, Mass.
Wormser, M.....Montclair, N. J
Wortendyke N. D.
Jersey City, N. J.
Worthington, C...New York City
Wrenn, J. F.....New York City
Wyckoff, C. R., Jr.,
Brooklyn, N. Y.
Wyman, A. M..East Orange, N. J.

Yates, J. J.....Plainfield, N. J.
Young, C. G.....New York City

ANNOUNCEMENTS

The House of the Society is open from 9 A. M. to 10 P. M., every day, except Sundays, Fourth of July, Thanksgiving Day, and Christmas Day.

MEETINGS

Wednesday, March 3d, 1909.—8.30 P. M.—Two papers will be presented for discussion, as follows: "The Action of Frost on Cement and Cement Mortar, Together with Other Experiments on These Materials," by Messrs. Ernest R. Matthews, and James Watson; and "The Bonding of New to Old Concrete," by E. P. Goodrich, M. Am. Soc. C. E.

These papers were printed in *Proceedings* for January, 1909.

Wednesday, March 17th, 1909.—8.30 P. M.—A paper entitled "Steel Sheeting and Sheet-Piling," by L. R. Gifford, Assoc. M. Am. Soc. C. E., will be presented for discussion.

This paper is printed in this number of *Proceedings.*

Wednesday, April 7th, 1909.—8.30 P. M.—A paper on "The Sixth Street Viaduct of Kansas City," by E. E. Howard, Assoc. M. Am. Soc. C. E., will be presented for discussion.

This paper is printed in this number of *Proceedings.*

Wednesday, April 21st, 1909.—8.30 P. M.—Two papers will be presented for discussion, as follows: "The Maximum Weights of Slow Freight Trains," by C. S. Bissell, M. Am. Soc. C. E.; and "Sampittic Surfacing," by W. W. Crosby, M. Am. Soc. C. E.

These papers are printed in this number of *Proceedings.*

ANNUAL CONVENTION

The Forty-first Annual Convention of the Society will be held at the Mount Washington Hotel, Bretton Woods, N. H., from July 6th to July 9th, 1909, inclusive.

The general arrangements for the Convention are in the hands of the following Committees:

COMMITTEE OF THE BOARD OF DIRECTION.

F. W. Hodgdon,

G. W. Tillson, Chas. Warren Hunt.

LOCAL COMMITTEE.

H. W. Hayes,

A. W. Dean, S. E. Tinkham,
H. D. Woods, J. F. Stevens,
George A. Kimball, J. W. Ellis.

PAPERS AND DISCUSSIONS

The Board of Direction has decided to issue *Transactions* in future as a quarterly publication. There will therefore be four volumes each year instead of two as heretofore.

There seems to be some misunderstanding about this matter, several members having protested on the ground that the present volumes are not too large, and that making them smaller would make reference to them more difficult, as well as increase the cost of binding.

It seems, therefore, necessary to explain that the decision of the Board is based on the fact that the amount of valuable material offered the Society for publication has so increased of late that a large part of it would have to be refused if only two volumes per annum were issued, and on the belief that the most effective way of increasing the usefulness of the Society to its Members and the Engineering Profession is to increase the volume of its Publications.

The decision was arrived at only after most careful consideration of the matter from every point of view, and it is hoped the result will be satisfactory to the membership.

It is also hoped that members and others who take part in the discussion of the papers presented will revise their remarks promptly, and that all written communications from those who cannot attend the meetings will be sent in at the earliest possible date after the issue of the paper in *Proceedings*. The issue of volumes of *Transactions* is dependent on the closing of discussions, and the co-operation of the membership will now be more necessary in this matter than heretofore, because four volumes are to be issued during the year instead of two, and, to accomplish this, a definite date of issue for each must be established. It is expected that the first volume for 1909 will be issued on or about March 31st, the second on June 30th, and the third and fourth on September 30th and December 31st, respectively.

All papers accepted by the Publication Committee are classified by the Committee with respect to their availability for discussion at meetings.

Papers, which, from their general nature, appear to be of a character suitable for oral discussion will be published as heretofore in *Proceedings*, and set down for presentation to a future meeting of the Society, and, on these, oral discussion, as well as written communications, will be solicited.

All papers which do not come under this heading, that is to say, those which, from their mathematical or technical nature, in the opinion of the Committee, are not adapted to oral discussion, will not be scheduled for presentation to any meeting. Such papers will be published in *Proceedings* in the same manner as those which are to be presented at meetings, but written discussions, only, will be requested for subsequent publication in *Proceedings* and with the paper in the volumes of *Transactions*.

PRIVILEGES OF ENGINEERING SOCIETIES EXTENDED TO MEMBERS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

Members of the American Society of Civil Engineers will be welcomed by the following Engineering Societies, both to the use of their Reading Rooms and at all Meetings:

American Institute of Mining Engineers, 29 West Thirty-ninth Street, New York City.

Associação dos Engenheiros Civis Portuguezes, Lisbon, Portugal.

Australasian Institute of Mining Engineers, Melbourne, Victoria, Australia.

Boston Society of Civil Engineers, 715 Tremont Temple, Boston, Mass.

Brooklyn Engineers' Club, 197 Montague Street, Brooklyn, N. Y.

Canadian Society of Civil Engineers, 877 Dorchester Street, Montreal, Que., Canada.

Civil Engineers' Club of Cleveland, 718 Caxton Building, Cleveland, Ohio.

Civil Engineers' Society of St. Paul, St. Paul, Minn.

Cleveland Institute of Engineers, Middlesbrough, England.

Engineers' and Architects' Club of Louisville, Ky., 303 Norton Building, Fourth and Jefferson Streets, Louisville, Ky.

Engineers' Club of Baltimore, Baltimore, Md.

Engineers' Club of Central Pennsylvania, Corner Second and Walnut Streets, Harrisburg, Pa.

Engineers' Club of Philadelphia, 1317 Spruce Street, Philadelphia, Pa.

Engineers' Club of St. Louis, 3817 Olive Street, St. Louis, Mo.

Engineers' Club of Toronto, 96 King Street, West, Toronto, Ont., Canada.

Engineers' Society of Western Pennsylvania, 803 Fulton Building, Pittsburg, Pa.

Institute of Marine Engineers, 58 Romford Road, Stratford, London, E., England.

Institution of Engineers of the River Plate, Buenos Aires, Argentine Republic.

Institution of Naval Architects, 5 Adelphi Terrace, London, W. C., England.

Junior Institution of Engineers, 39 Victoria Street, Westminster, S. W., London, England.

Koninklijk Instituut van Ingenieurs, The Hague, The Netherlands.

Louisiana Engineering Society, 604 Tulane-Newcomb Building. New Orleans, La.

Memphis Engineering Society, Memphis, Tenn.

Midland Institute of Mining, Civil and Mechanical Engineers, Sheffield, England.

Montana Society of Engineers, Butte, Montana.

North of England Institute of Mining and Mechanical Engineers, Newcastle-upon-Tyne, England.

Oesterreichischer Ingenieur- und Architekten-Verein, Eschenbachgasse 9, Vienna, Austria.

Pacific Northwest Society of Engineers, 617-618 Pioneer Building, Seattle, Wash.

Rochester Engineering Society, Rochester, N. Y.

Sachsischer Ingenieur- und Architekten-Verein, Dresden, Germany.

Sociedad Colombiana de Ingenieros, Bogota, Colombia.

Societe des Ingenieurs Civils de France, 19 Rue Blanche, Paris, France.

Society of Engineers, 17 Victoria Street, Westminster, S. W., England.

Svenska Teknologföreningen, Brunkebergstorg 18, Stockholm, Sweden.

Tekniske Forening, Vestre Boulevard 18–1, Copenhagen, Denmark.

Western Society of Engineers, 1737 Monadnock Block, Chicago, Ill.

SEARCHES IN THE LIBRARY

In January, 1902, the Secretary was authorized to make searches in the Library, upon request, and to charge therefor the actual cost to the Society for the extra work required. Since that time many searches have been made, and bibliographies and other information on special subjects furnished.

The resulting satisfaction, to the members, who have made use of the resources of the Society in this manner, has been expressed frequently, and leaves little doubt that, if it were generally known to the membership that such work would be undertaken, many would avail themselves of it.

The cost is trifling compared with the value of the time of an engineeer who looks up such matters himself, and the work can be performed quite as well, and much more quickly, by persons familiar with the Library.

In asking that such work be undertaken, members should specify clearly the subject to be covered, and whether references to general books only are desired, or whether a complete bibliography, involving search through periodical literature, is desired.

In reference to this work the Appendix* to the Annual Report of the Board of Direction for the year ending December 31st, 1906, contains a summary of all searches made to that date.

* *Proceedings*, Vol. XXXIII, p. 20 (January, 1907).

ACCESSIONS TO THE LIBRARY

(From January 13th to February 9th, 1909)

DONATIONS*

HYDRAULIC TABLES.

The Elements of Gagings and the Friction of Water Flowing in Pipes, Aqueducts, Sewers, etc., as Determined by the Hazen and Williams Formula and the Flow of Water over Sharp-Edged and Irregular Weirs, and the Quantity Discharged as Determined by Bazin's Formula and Experimental Investigations upon Large Models. By Gardner S. Williams and Allen Hazen, Members, Am. Soc. C. E. Second Edition, Revised and Enlarged. Cloth, 9 x 6 in., illus., 6 + 104 pp. New York, John Wiley & Sons, 1909. $1.50.

In this edition all errors in the original text and tables, developed by the continuous use of the book for over three years are stated to have been corrected. The preface also states that, the changes in this edition are nearly all confined to that part of the book devoted to the flow of water over weirs, where some new matter on submerged weirs is presented in the text and the table of discharge by Bazin's formula has been extended to cover variations of head by 0.01 ft. from zero to 6 ft. A table of discharge of high weirs has also been added and these additions, with a new and more correct title-page and an extended table of contents, are intended to increase materially the usefulness of the work. The Contents are: Introduction; Formula Used; Increasing Friction with Age, How Computed and Indicated in the Tables; Observations of Flow in Cast-Iron Pipe; Observations of Flow in Riveted Steel Pipe; Observations of Flow in Wooden-Stave Pipe; Observations of Flow in Rectangular Wooden Pipe; Observations of Flow in Cement Pipe; Observations of Flow in Wrought-Iron Pipe; Observations of Flow in Galvanized-Iron Pipe; Observations of Flow in Brass Pipe; Observations of Flow in Lead Pipe; Observations of Flow in Glass Pipe; Observations of Flow in Fire-Hose; Observations of Flow in Open Conduits; Observations of Flow in Aqueducts; Observations of Flow in Brick Sewers; Observations of Flow in Canals; Table of Flow in Small Brass Pipes; Table of Flow in Wrought-Iron Pipes; Table of Flow in Hose and Pipes; Table of Flow in Pipes, 4 to 144 in. Diameter. Relative Discharging Capacity of Aqueducts. Table of Flow in Aqueducts; Sewers, Table of Slopes Required to Produce Certain Velocities; Tile Sewer Table; Circular Brick Sewer Table; Decrease in Carrying Capacity of Cast-Iron Pipe with Age; Comparison of Results with those of Coffin and Weston; Metric Pipe Table. Loss of Head in Venturi Meter; Underdrains, for Sand-Filters; Flow Over Weirs; Sharp-Edged Weirs, Discussion; Sharp-Edged Weirs, Tables of Discharge; Low Heads, and Contractions and Very High Weirs, Discussion; High Heads, Table of Discharge; High Weirs, Table of Discharge; Flat Crest and Other Weirs, Discussion; Submerged Weirs, Discussion; Multipliers for Flat Crest Weirs; Multipliers for Trapezoidal Weirs; Multipliers for Triangular Weirs; Multipliers for Compound Weirs; Multipliers for Complex Weirs.

ARTIFICIAL WATERWAYS AND COMMERCIAL DEVELOPMENT.

(With a History of the Erie Canal.) By A. Barton Hepburn. Cloth, 7½ x 5 in., 7 + 115 pp. New York, The Macmillan Company, 1909. $1.00.

The purpose of this volume, as declared by the author, is to place before the public, in concise form, the salient facts as to artificial waterways and their relation to commercial development. A general review of the canal systems of the world is given, with the history of the Erie Canal as typical of all. The Contents are: The World's Canals; The Canal System of New York, I, The Period of Inception, II, The Period of Development, III, The Present Conditions, IV, The Competition against New York City; The Panama Canal; The Waterways Question and Conservation of Our Resources; Appendix (Statistical Tables) *A*, Relating to New York Canals, *B*, Relating to Commerce of New York City, *C*, Miscellaneous; Index.

APPLIED MECHANICS FOR ENGINEERS.

A Text-Book for Engineering Students. By E. L. Hancock. Cloth, 7½ x 5 in., illus., 11 + 385 pp. New York, The Macmillan Company, 1909. $2.00 net.

This work is intended as a text-book for engineering students of the Junior year, and the subject-matter, except that contained in a few chapters, is stated to be

*Unless otherwise specified, books in this list have been donated by the publisher.

such as is usually covered in one semester. Each new principle developed is followed, it is stated, by a number of applications and each problem deals with matters that directly concern the engineer, and constitute practical engineering work. The Chapter headings are: Definitions; Concurrent Forces; Parallel Forces; Center of Gravity; Couples; Non-concurrent Forces; Moment of Inertia; Flexible Cords; Rectilinear Motion; Curvilinear Motion; Rotary Motion; Dynamics of Machinery; Work and Energy; Friction; Impact; Appendix I, Hyperbolic Functions, Tables; Appendix II, Logarithms of Numbers; Appendix III, Trigonometric Functions, Tables; Appendix IV, Squares, Cubes, Square Roots, etc., of Numbers; Appendix V, Conversion Tables; Index.

TIMBER.

By J. R. Baterden. Cloth, 8½ x 5½ in., illus., 10 + 351 pp. London, Archibald Constable & Co., Ltd., 1908. 6 shillings net.

The author states that this book is intended to be essentially a practical work, with botany only incidentally touched upon. Only those timbers most generally used, either in their native districts, or in the general timber trade, are discussed, together with some likely to come into the market before long. Those timbers most used have been dealt with at greatest length. No information is given, the preface states, of which the author is not either certain from his own knowledge or has not gained from authentic sources. There is an appendix, giving sample cargoes of timber and showing the great variety and large quantities of timber landed in Great Britain, and a bibliography of one and one-half pages. The Contents are: Timber; The World's Forest Supply; Quantities of Timber Used; Timber Imports into Great Britain; European Timber; Timber of United States and Canada; Timbers of South America, Central America, and West India Islands; Timbers of India, Burma, and Andaman Islands; Timber of the Straits Settlements, Malay Peninsula, Japan, and South and West Africa; Australian Timbers; Timbers of New Zealand and Tasmania; Causes of Decay and Destruction of Timber; Seasoning and Impregnation of Timber; Defects in Timber, and General Notes; Strength and Testing of Timber; "Figure" in Timber; Appendix; Bibliography; Index.

Gifts have also been received from the following:

Am. Inst. of Elec. Engrs. 1 bound vol.
Am. Mathematical Soc. 1 pam.
Am. Soc. for Testing Materials. 1 vol.
Am. Soc. Mech. Engrs. 1 bound vol.
Architectural Inst. of Canada. 1 pam.
Atlantic City, N. J.-Water Dept. 2 pam.
Belgium-Ministère des Chemins de Fer Postes et Télégraphes. 1 vol.
Bengal, India-Irrig. Dept. 1 pam.
Bombay-Public Works Dept. 1 bound vol.
Buck, H. R. 1 bound vol., 8 pam.
Central London Ry. Co. 1 pam.
Colorado-Agri. Exper. Station. 2 pam.
Connecticut-R. R. Commrs. 1 bound vol.
Connecticut-Shell-fish Comm. 1 pam.
Cornell Univ. 1 vol.
Cuba-Secretaria de Obras Publicas. 1 vol.
Delft Technische Hoogeschool. 2 pam.
Deutsch-Amer. Tech. Verband. 2 pam.
Fall River, Mass.-City Engr. 6 pam.
Ferraz, L. C. 1 pam.
Germany-Kaiserliche General Direktion der Eisenbahnen in Elsass-Lothringen. 1 vol.
Great Eastern Ry. Co. 1 pam.
Great Northern & City Ry. Co. 1 pam.
Harvard Univ. 1 bound vol.
Hodgdon, Frank W. 1 bound vol.
Herschel, Clemens. 2 pam.
Illinois-State Water Survey. 1 pam.
Institution of Civ. Engrs. 1 bound vol., 3 pam.
Institution of Engrs. and Shipbuilders of Scotland. 1 bound vol.
Inter. Assoc. of Mun. Electricians. 1 bound vol., 1 pam.
Japan-Imperial Govt. Rys. 2 pam., 1 map.
Johnson, G. A. 1 pam.
Lake Mohonk Conference of Friends of the Indian and other Dependent Peoples. 1 pam.
Liverpool Eng. Soc. 1 bound vol.
Liverpool Overhead Ry. Co. 1 pam.
London, Brighton & South Coast Ry. Co. 1 pam.
London, Tilbury & Southend Ry. Co. 1 pam.
Madras, India-Public Works Dept. 1 pam.
Met. Ry. Co. 1 pam.
Min. Soc. of Nova Scotia. 1 vol.
New York City-Art Comm. 1 pam.
New York City-Board of Water Supply. 4 pam.
New York City-Comptroller. 1 pam.
New York City-Dept. of Health. 1 pam.
New York State-Public Service Comm. 3 pam.
New York-State Engr. and Surv. 1 bound vol.
Ontario, Canada-Bureau of Mines. 4 pam.
Ontario, Canada-Registrar-Gen. 1 pam.
Oregon-Conservation Comm. 1 pam.
Peru-Ministerio de Fomento. 1 vol.
Rensselaer Polytechnic Inst. 1 vol.
Rochester, N. Y.-City Engr. 1 pam.
R. Universita Romana, Scuola d'Applicazione per gl'Ingegneri. 1 vol.
Sherrerd, M. R. 1 pam.

South Dakota-State Engr. 1 bound vol.
South Eastern Ry. Co. 2 pam.
Southern Pacific Co. 1 pam.
Taff Vale Ry. Co. 1 pam.
Thomes, E. H. 1 pam.
Thomson, T. K. 1 pam.
U. S.-Bureau of the Census. 1 bound vol.
U. S.-Chief of Engrs. 7 pam.
U. S.-Coast and Geodetic Survey. 1 bound vol.
U. S.-Engr. School. 1 pam.
U. S.-Geol. Survey. 13 pam.
U. S.-Interstate Commerce Comm. 1 pam.
U. S.-Lighthouse Board. 1 bound vol.
U. S.-Naval Observatory. 1 pam.
U. S.-Office of Exper. Stations. 1 bound vol.
U. S.-Office of the Library and Naval War Records. 1 bound vol., 7 pam.
U. S.-Weather Bureau. 1 bound vol.
Univ. of Illinois-Agri. Exper. Station. 3 pam.
Univ. of Pennsylvania. 1 vol.
Univ. of Texas. 2 pam., 1 map.
Victoria, Australia-Ry. Commrs. 1 vol.
Webster, W. R. 1 pam.
Yale Univ. 1 vol.

BY PURCHASE

Report with Regard to Civic Affairs in the City of Cedar Rapids, Iowa, with Recommendations for City Improvement and Beautification. By C. M. Robinson. The Torch Press. Cedar Rapids, 1908.

Repertorium der Technischen Journal-literatur. Herausgegeben im Kaiserlichen Patentamt, Jahrgang 1907. Carl Heymanns, Berlin, 1908.

SUMMARY OF ACCESSIONS

From January 13th to February 9th, 1909.

Donations (including 7 duplicates)	143
By purchase	2
Total	145

MEMBERSHIP

ADDITIONS

(January 13th to February 9th, 1909.)

MEMBERS		Date of Membership.
ALDEN, HERBERT CLARENDON. Asst. Engr., Bureau of Sewers, Jerome Ave. and Macombs Rd., Borough of the Bronx, New York City	Assoc. M. M.	Mar. 1, 1893 Feb. 2, 1909
BULL, GEORGE MAIRS. Hydr. Engr., Denver Reservoir Irrig. Co., 705 Ideal Bldg., Denver, Colo		Feb. 2, 1909
HARRINGTON, FERDINAND FINNEY. Bridge Engr., The Virginian Ry. Co., National Bank of Commerce Bldg., Norfolk, Va	Assoc. M. M.	May 3, 1899 Feb. 2, 1909
HAUGH, JAMES CHARLES. Res. Engr., New Orleans & North Eastern R. R., Press and Levee Sts., New Orleans, La.		Feb. 2, 1909
HORNSBY, ROBERT WRIGHT. Casilla 162, Valparaiso, Chili..		Nov. 4, 1908
KNIGHT, RICHARD WARREN. Contr. Engr., McClintic-Marshall Constr. Co., Park Bldg., Pittsburg, Pa		Feb. 2, 1909
MANAHAN, ELMER GOVE. With Hering & Fuller, 170 Broadway, New York City	Assoc. M. M.	Oct. 7, 1903 Feb. 2, 1909
MARR, WILLIAM WALTER. 1893 Gladys Ave., Chicago, Ill..		Feb. 2, 1909
MAXIMOFF, SERGIUS PAVLOVITCH. Engr. to the Imperial Russian Govt.; Asst. at the Imperial Inst. of Public Ways at St. Petersburg, Zabalkanski pr. 21, St. Petersburg, Russia	Assoc. M. M.	April 1, 1903 Sept. 1, 1908
MOORE, JOHN WILLIAM. 831 Morgan St., Rushville, Ind....		Feb. 2, 1909
NEWTON, RALPH EELLS. Pres., Newton Engineering Co., 434 Jackson St., Milwaukee, Wis	Assoc. M. M.	Jan. 8, 1902 Feb. 2, 1909
WALMSLEY, WALTER NEWBOLD. Gen. Mgr., São Paulo Tramway, Light & Power Co., Ltd., São Paulo, Brazil....		Dec. 1, 1908

ASSOCIATE MEMBERS		
BLAMPHIN, ARTHUR MERRICK NEWBERRY. Asst. City Engr., Room 19, City Hall, New Orleans, La	Jun. Assoc. M.	Dec. 3, 1907 Feb. 2, 1909
BROWN, ALFRED THOMAS. Dept. of Highways, Borough of the Bronx, Cor. 177th St. and 3d Ave., New York City	Jun. Assoc. M.	Nov. 1, 1904 Jan. 5, 1909
EDWARDS, CHARLES MILTON. First Asst. Engr., Dept. of N. Y. State Engr., 43 Triangle Bldg., Rochester, N. Y.		Sept. 2, 1908
FOWLER, ROBERT LAMBERT. 155 State St., Perth Amboy, N. J		Feb. 2, 1909

ASSOCIATE MEMBERS (*Continued*).

		Date of Membership.
GODDARD, HERBERT WILLARD. Engr. and Contr. (Cessna & Goddard), 178 Devonshire St., Boston, Mass.	Jun.	Jan. 3, 1905
	Assoc. M.	Jan. 5, 1909
LAMBERT, BYRON JAMES. Prof. of Structural Eng., State Univ. of Iowa, Iowa City, Iowa		Jan. 5, 1909
LEEFE, FREDERICK EWBANK. Care, U. S. Engr. Office, Baltimore, Md		May 6, 1908
MACNAUGHTON, ERNEST BOYD. Care, MacNaughton, Raymond & Lawrence, Archts. and Engrs., Portland, Ore.		Oct. 7, 1908
PAIGE, JASON. Am. Bridge Co. of N. Y., 1308 Commercial National Bank Bldg., Chicago, Ill.	Jun.	Feb. 5, 1907
	Assoc. M.	Feb. 2, 1909
PIERCE, GEORGE ABEL. Asst. Engr., Champaign County, 120 East Church St., Urbana, Ohio		Jan. 5, 1909
SANGER, EDMUND PHIPPS. 301 S. 4th Ave., Mt. Vernon, N. Y.		Feb. 2, 1909
SMITH, ALBERT ORANGE. Civ. and Landscape Engr., Port Jefferson, N. Y.	Jun.	May 2, 1905
	Assoc. M.	Feb. 2, 1909
TALLMAN, LEROY. Portsmouth, R. I.		Feb. 2, 1909
TAYLOR, GRANVILLE LEWIS. Engr., Pittsburg Plant, McClintic-Marshall Const. Co.; Res., 119 Trenton Ave., Wilkinsburg, Pa.		Feb. 2, 1909
TÖNNESEN, FRIDTHJOV LAURITZ MARTIN. Hallingbury, Camp P. O., Jamaica		June 3, 1908

ASSOCIATE

BROMLEY, ALBERT HENRY, JR. Concrete Engr., 5113 Market St., Philadelphia, Pa.	Feb. 2, 1909

JUNIORS

BRENNAN, JOSEPH LAWRENCE. Brown Station, N. Y.	Jan. 5, 1909
DUBOIS, GUSTAVO ADOLFO. Constitucion 22, Matanzas, Cuba	Sept. 1, 1908
GREEN, CLARENCE JASPER. 907 Monroe St., Ann Arbor, Mich.	Sept. 1, 1908
GRINDROD, IRVIN SUTTON. 33d and Clearfield Sts., Philadelphia, Pa.	Feb. 2, 1909
HATHAWAY, CLIFFORD MURRAY. Draftsman, R. I. State Board of Public Roads, 50 Park St., Providence, R. I.	Sept. 1, 1908
PRITCHARD, JOHN CHARLES. Deputy State Highway Engr. of Mo., Columbia, Mo.	Sept. 1, 1908
RUSSELL, ALEXANDER STUART. Asst. Engr., Bureau of Public Works, Manila, Philippine Islands	Oct. 6, 1908
TILLIT, PEDRO ERNESTO. P. O. Box 1207, Lima, Peru	Oct. 6, 1908
WILLIAMS, HAROLD S. Caldwell, Idaho	Sept. 1, 1908
WILMOT, JAMES. Office of Public Roads, Washington, D. C.	Sept. 1, 1908

RESIGNATIONS

MEMBER	Date of Resignation.
MICHELL, THEOPHILUS	December 31, 1908
ASSOCIATE MEMBER	
BURNS, JAMES FERGUSON	February 2, 1909
JUNIORS	
BRASLOW, BARNETT	February 2, 1909
O'NEAL, ALFRED MOORE, JR	February 2, 1909

DEATHS

ADGATE, GEORGE. Elected Member, April 1st, 1896; died January 25th, 1909.

AMBROSE, WILLIAM CREELMAN. Elected Member, April 4th, 1888; died January 3d, 1909.

BOYLE, OLIN MCCLINTOCK, JR. Elected Junior, March 5th, 1907; died August 19th, 1908.

CRAIGHILL, WILLIAM PRICE (*Past-President*). Elected Member, October 7th, 1885; Honorary Member, March 23d, 1896; died January 18th, 1909.

GRAHAM, JOSEPH MARSHALL. Elected Member, April 4th, 1900; died February 4th, 1909.

QUINETTE DE ROCHEMONT, *Baron* EMILE THEODORE. Elected Member, January 3d, 1894; died December 8th, 1908.

MONTHLY LIST OF RECENT ENGINEERING ARTICLES OF INTEREST

(January 12th, to February 8th, 1909.)

NOTE.—*This list is published for the purpose of placing before the members of the Society, the titles of current engineering articles, which can be referred to in any available engineering library, or can be procured by addressing the publication directly, the address and price being given wherever possible.*

LIST OF PUBLICATIONS

In the subjoined list of articles, references are given by the number prefixed to each journal in this list:

(1) *Journal*, Assoc. Eng. Soc., 31 Milk St., Boston, Mass., 30c.
(2) *Proceedings*, Engrs. Club of Phila., 1317 Spruce St., Philadelphia, Pa.
(3) *Journal*, Franklin Inst., Philadelphia, Pa., 50c.
(4) *Journal*, Western Soc. of Engrs., Monadnock Blk., Chicago, Ill.
(5) *Transactions*, Can. Soc. C. E., Montreal, Que., Canada.
(6) *School of Mines Quarterly*, Columbia Univ., New York City, 50c.
(7) *Technology Quarterly*, Mass. Inst. Tech., Boston, Mass., 75c.
(8) *Stevens Institute Indicator*, Stevens Inst., Hoboken, N. J., 50c.
(9) *Engineering Magazine*, New York City, 25c.
(10) *Cassier's Magazine*, New York City, 25c.
(11) *Engineering* (London), W. H. Wiley, New York City, 25c.
(12) *The Engineer* (London), International News Co., New York City, 35c.
(13) *Engineering News*, New York City, 15c.
(14) *The Engineering Record*, New York City, 12c.
(15) *Railroad Age Gazette*, New York City, 15c.
(16) *Engineering and Mining Journal*, New York City, 15c.
(17) *Electric Railway Journal*, New York City, 10c.
(18) *Railway and Engineering Review*, Chicago, Ill., 10c.
(19) *Scientific American Supplement*, New York City, 10c.
(20) *Iron Age*, New York City, 10c.
(21) *Railway Engineer*, London, England, 25c.
(22) *Iron and Coal Trades Review*, London, England, 25c.
(23) *Bulletin*, American Iron and Steel Assoc., Philadelphia, Pa.
(24) *American Gas Light Journal*, New York City, 10c.
(25) *American Engineer*, New York City, 20c.
(26) *Electrical Review*, London, England.
(27) *Electrical World*, New York City, 10c.
(28) *Journal*, New England Water-Works Assoc., Boston, Mass., $1.
(29) *Journal*, Royal Society of Arts, London, England, 15c.
(30) *Annales des Travaux Publics de Belgique*, Brussels, Belgium.
(31) *Annales* de l'Assoc. des Ing. Sortis des Ecoles Spéciales de Gand, Brussels, Belgium.
(32) *Mémoires et Compte Rendu des Travaux*, Soc. Ing. Civ. de France, Paris, France.
(33) *Le Génie Civil*, Paris, France.
(34) *Portefeuille Economiques des Machines*, Paris, France.
(35) *Nouvelles Annales de la Construction*, Paris, France.
(37) *Revue de Mécanique*, Paris, France.
(38) *Revue Générale des Chemins de Fer et des Tramways*, Paris, France.
(41) *Modern Machinery*, Chicago, Ill., 10c.
(42) *Proceedings*, Am. Inst. Elec. Engrs., New York City, 50c.
(43) *Annales des Ponts et Chaussées*, Paris, France.
(44) *Journal*, Military Service Institution, Governors Island, New York Harbor, 50c.
(45) *Mines and Minerals*, Scranton, Pa., 20c.
(46) *Scientific American*, New York City, 8c.
(47) *Mechanical Engineer*, Manchester, England.
(48) *Zeitschrift*, Verein Deutscher Ingenieure, Berlin, Germany.
(49) *Zeitschrift für Bauwesen*, Berlin, Germany.
(50) *Stahl und Eisen*, Düsseldorf, Germany.
(51) *Deutsche Bauzeitung*, Berlin, Germany.
(52) *Rigasche Industrie-Zeitung*, Riga, Russia.
(53) *Zeitschrift*, Oesterreichischer Ingenieur und Architekten Verein, Vienna, Austria.
(54) *Transactions*, Am. Soc. C. E., New York City, $4.
(55) *Transactions*, Am. Soc. M. E., New York City, $10.
(56) *Transactions*, Am. Inst. Min. Engrs., New York City, $5.

(57) *Colliery Guardian*, London, England.
(58) *Proceedings*, Eng. Soc. W. Pa., 803 Fulton Bldg., Pittsburg, Pa., 50c.
(59) *Transactions*, Mining Inst. of Scotland, London and Newcastle-upon-Tyne, England.
(60) *Municipal Engineering*, Indianapolis, Ind., 25c.
(61) *Proceedings*, Western Railway Club, 225 Dearborn St., Chicago, Ill., 25c.
(62) *Industrial World*, 59 Ninth St., Pittsburg, Pa.
(63) *Minutes of Proceedings*, Inst. C. E., London, England.
(64) *Power*, New York City, 20c.
(65) *Official Proceedings*, New York Railroad Club, Brooklyn, N. Y., 15c.
(66) *Journal of Gas Lighting*, London, England, 15c.
(67) *Cement and Engineering News*, Chicago, Ill., 25c.
(68) *Mining Journal*, London, England.
(70) *Engineering Review*, New York City, 10c.
(71) *Journal*, Iron and Steel Inst., London, England.
(73) *Electrician*, London, England, 18c.
(74) *Transactions*, Inst. of Min. and Metal., London, England.
(75) *Proceedings*, Inst. of Mech. Engrs., London, England.
(76) *Brick*, Chicago, Ill., 10c.
(77) *Journal*, Inst. Elec. Engrs., London, England.
(78) *Beton und Eisen*, Vienna, Austria.
(79) *Forscherarbeiten*, Vienna, Austria.
(80) *Tonindustrie Zeitung*, Berlin, Germany.
(81) *Zeitschrift für Architektur und Ingenieurwesen*, Wiesbaden, Germany.
(82) *Dinglers Polytechnisches Journal*, Berlin, Germany.
(83) *Progressive Age*, New York City, 15c.
(84) *Le Ciment*, Paris, France.
(85) *Proceedings*, Am. Ry. Eng. and M. of W. Assoc., Chicago, Ill.
(86) *Engineering-Contracting*, Chicago, Ill., 10c.
(87) *Roadmaster and Foreman*, Chicago, Ill., 10c.
(88) *Bulletin* of the International Ry. Congress Assoc., Brussels, Belgium.
(89) *Proceedings*, Am. Soc. for Testing Materials, Philadelphia, Pa.
(90) *Transactions*, Inst. of Naval Archts., London, England.
(91) *Transactions*, Soc. Naval Archts. and Marine Engrs., New York City.
(92) *Bulletin*, Soc. d'Encouragement pour l'Industrie Nationale, Paris, France.
(93) *Revue de Métallurgie*, Paris, France, 4 fr. 50.
(94) *The Boiler Maker*, New York City, 10c.
(95) *International Marine Engineering*, New York City, 20c.

LIST OF ARTICLES

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The Curzon Bridge at Allahabad.* Robert Richard Gales, M. Inst. C. E. (63) Vol. 174.

The Netravati Bridge at Mangalore.* Archibald Scott Napier, M. Inst. C. E. (63) Vol. 174.

The King Edward VII Bridge, Newcastle-on-Tyne.* Frank William Davis and Cyril Reginald Sutton Kirkpatrick, Assoc. MM. Inst. C. E. (63) Vol. 174.

The New Bridge Crossing the Mississippi River at Clinton, Ia.; Chicago & Northwestern Ry.* F. H. Bainbridge. (13) Jan. 21.

Rio Conchos Bridge of the Kansas City, Mexico & Orient.* W. W. Colpitts, M. Am. Soc. C. E. (15) Jan. 22.

The Rocky River Concrete Bridge near Cleveland, O.* (14) Jan. 23.

Cost of Concrete Bridges. Henry H. Quimby. (Paper read before the National Assoc. of Cement Users.) (14) Jan. 23.

Cost of Small Concrete Piers.* (14) Jan. 23.

The Footwalks of the Blackwell's Island Bridge.* F. W. Abbott, M. Am. Soc. C. E., and J. P. H. Perry. (13) Jan. 28.

Live Loads and Working Stresses in Railway Bridges.* Conrad Gribble, A. M. I. C. E. (10) Feb.

A Comparison of Bridge Waterways.* Daniel B. Luten. (Paper read before the Ind. Eng. Soc.) (60) Feb.

Methods and Cost of Constructing a Plate Girder Bridge with Concrete Piers.* (86) Feb. 3.

An Ornamental Bascule Bridge; Trunnion Bascule across Coopers Creek, at Federal St., Camden, N. J.* (13) Feb. 4.

Conventional Live-Load Positions for Maximum Stresses in Arches. Wm. Cain, M. Am. Soc. C. E. (13) Feb. 4.

Central of New Jersey Bridge Renewal at Bethlehem.* (15) Feb. 5.

Les Ponts Modernes.* José-Eugenio Ribera. (From *Revista de Obras Publicas.*) (43) May. 1908.

Le Pont Neuf, Pont-Levant à Manœuvre Electrique au Port de Cette.* Herrmann. (43) May, 1908.

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Bridge—(Continued).

Pont sur la Maumee (Ohio).* (84) Dec.
Ponts en Poutrelles Enrobées.* Siegler. (38) Jan.
Pont en Béton Armé sur le Rhône, à Pyrimont (Ain et Haute-Savoie).* G. Espitallier. (33) Jan. 16.
Strassenbrücke über die Memel bei Tilsit, Königin-Luisen-Brücke.* Dieckmann. (49) Vol. 1-3, 1909.
Eine neue Graphische Berechnungsmethode für Beiderseits Eingespannte Träger mit Veränderlichem Trägheitsmoment. Rich. Wuczkowski. (78) Jan. 8.

Electrical.

Report of Committee on the Magnetic Testing of Iron and Steel. (Amer. Soc. for Testing Materials.)* (89) Vol. 8.
Uniformity in Magnetic Testing and in the Specification of Magnetic Properties.* Chas. W. Burrows. (89) Vol. 8.
The Development of the Electric Piano Player.* John F. Kelly. (3) Jan.
The Telegraphone. Charles K. Fankhauser. (3) Jan.
The Electrical Qualities of Porcelain, with Special Reference to Dielectric Losses.* H. F. Haworth. (Abstract of paper read before the Royal Soc.) (73) Jan. 1.
The Hemsjö Power Company, Sweden.* P. Frenell. (73) Serial beginning Jan. 8.
The Post Office Radio-Telegraph Station at Bolt Head.* (27) Jan. 8.
System of the Rochester Railway & Lighting Company.* (27) Jan. 14.
Factors Governing the Space Utilization of Electro-magnetic Windings.* Charles R. Underhill. (27) Jan. 14.
A Directive System of Wireless Telegraphy. E. Bellini and A. Tosi. (Abstract of paper read before the Physical Soc.) (73) Jan. 15.
Deformation of Pressure Curves due to Load in Alternating Current Generators. Egon Siedek. (Abstract from *Elek. u. Maschinenbau.*) (73) Jan. 15.
Insulation of High-Tension Transmission Lines.* F. S. Denneen. (Abstract of paper read before the Central Elec. Club.) (73) Jan. 15.
Station No. 3 of the Rochester Railway & Light Company.* (27) Jan. 21.
The Influence of Frequency on the Equivalent Circuits of Alternating-Current Transmission Lines.* A. E. Kennelly. (27) Jan. 21
Representation of Synchronous Converter Phenomena.* Olin J. Ferguson. (27) Jan. 21.
The Improvement of Power Factor in Alternating-Current Systems.* Miles Walker. (Abstract of paper read before the Inst. Elec. Engrs.) (73) Jan. 22.
Korn's Apparatus for Photographic Transmission.* Louis Dubois. (73) Jan. 22.
Electricity and its Application to the Reduction of Waste.* A. B. Gridley. (Abstract of paper read before the Cleveland and Durham Eng. Soc.) (73) Jan. 22.
Metallic Filament Lamps.* W. A. Barnes. (Abstract of paper read before the Manchester Assoc. of Engrs.) (47) Serial beginning Jan. 22.
The Power System of the Rochester Railway & Light Company.* (17) Jan. 23; (27) Jan. 28.
Fluctuations in the Speed of the Rotors of Alternators Operating in Parallel.* George H. Shepard. (27) Jan. 28.
A New System of Wireless Telephony.* A. Carletti. (73) Jan. 29.
Parallel Operation of Alternators.* E. Rosenberg. (Abstract of paper read before the Inst. Elec. Engrs.) (73) Serial beginning Jan. 29.
Sending Pictures by Wireless.* Alfred Gradenwitz. (19) Jan. 30.
Prime Movers.* Charles P. Steinmetz. (42) Feb.
Electric Heating. W. S. Hadaway, Jr. (42) Feb.
Length and Voltage of Transmission Lines.* Alton D. Adams. (10) Feb.
Electric Motors in a Brewery.* William Kelly. (27) Feb. 4.
Comparative Tests of Transformers.* A. C. Scott. (27) Feb. 4.
Cost of Energy in Nine Central Stations Operating Mixed Equipment. (27) Feb. 4.
Note sur le Télé-Stéréographe.* E. Belin. (92) Dec.
Grue Electrique Pivotante Fixe de 30-10 Tonnes du Port de Nice.* Ch. Dantin. (33) Jan. 9.
Die Gleichstrom-Querfeldmaschine und Ihre Anwendungen, Insbesondere für Elektrische Zugbeleuchtung.* Richard Neumann. (48) Jan. 23.

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The Efficiency of Marine Engines and Propellers.* J. Hamilton Gibson. (12) Jan. 29.
Magnetic Survey Yacht *Carnegie.** (95) Feb.
Piping up a Merchant Vessel for Steam Heat.* Allan Dale. (95) Feb.

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The Development and Present Status of the Experimental Model-Towing Basin.* H. A. Everett. (95) Feb.
A New Reversing Motor for Launches and Yachts.* G. O. M. Olsson. (Comp. from *Teknisk Tidskrift.*) (95) Feb.
Application du Moteur Diesel à la Navigation.* A. Bochet. (32) Nov.
La Traction Electrique des Bateaux sur les Canaux.* P. du Bousquet. (32) Nov.
Der Doppelschrauben-Personen- und Frachtdampfer *Prinz Friedrich Wilhelm.* (48) Serial beginning Jan. 2.

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The Temperature Gradient in De Laval Steam-Nozzles.* Cyril Batho. (63) Vol. 174.
Method of Obtaining a Truly Circular and Uniform Chill in Rolls.* Thomas D. West. (89) Vol. 8.
The Influence of Fine Grinding on the Physical Properties of Portland Cement. Richard K. Meade. (89) Vol. 8.
Testing Lubricating Oils. Henry Souther. (89) Vol. 8.
Special Features of a Recently Installed 600 000-Lb. Universal Testing Machine.* Thorsten Y. Olsen. (89) Vol. 8.
New Forms of Pendulum Testing Machines.* Thorsten Y. Olsen. (89) Vol. 8.
Commercial Results in the Purchase of Coal on Specifications.* J. E. Woodwell. (89) Vol. 8.
The Use of the Extensometer in Commercial Work.* T. D. Lynch. (89) Vol. 8.
The Direct Production of Copper Tubes, Sheets and Wire.* Sherard O. Cowper-Coles. (75) July.
The Evolution and Methods of Manufacture of Spur-Gearing.* Thomas Humpage. (75) July.
Inclined Retort Coal- and Coke-Handling Plant at Bristol. William Stagg. (75) July.
The Destructive Agencies Acting upon Refractories. A. V. Bleininger. (76) Jan.
Salt-Glazed Blue Brick. D. Cobb, A. M. I. M. E. (76) Jan.
Modern Practice in Making Dry Press-Brick. John J. Moroney. (76) Jan.
An "Up-to-the-Minute" Yard, Scott System and Gas-Fired Kiln at Spinks Plant, Covington, Kentucky.* (76) Jan.
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The Collapse of Tubes under External Pressure.* S. E. Slocum. (11) Jan. 8.
Temperley Transporters.* (11) Serial beginning Jan. 8; (57) Jan. 15.
Rock Drill Making and Sharpening by Machine and Treatment of Drill Steel.* W. E. Kimber. (Paper read before the Transvaal Inst. of Mech. Engrs.) (22) Jan. 8.
The Coal-Screening and Conveying Plant at the Collieries of Messrs. Burn-Yeat Brown and Co., Nine Mile Point, near Newport, Monmouth.* (57) Jan. 8.
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A. B. C. of Combustion in Boiler-Settings. Jacques Abady, M. Inst. M. E. (Paper read before the Soc. Mech. and Elec. Engrs.) (66) Jan. 5.
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The Commercial Truck vs. the Horse.* Benjamin Rogers. (46) Jan. 16.
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Hampton Power Plant of the D., L. & W. R. R.* Warren O. Rogers. (64) Jan. 19.
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Coal Briquetting, with Special Reference to the Briquetting Machine Built and Used by the Standard Fuel Company, Birmingham, Ala.* R. M. Hale. (20) Jan. 28.
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Trials of Two Pairs of Compound Engines.* Michael Longridge, M. Inst. C. E. (From Report of Chf. Engr., the Brit. Engine, Boiler and Elec. Insurance Co., Ltd.) (47) Serial beginning Jan. 29.
Experiments with Rolling Mill Plant.* F. Carleton Anderson. (Abstract of paper read before the Manchester Assoc. of Engrs.) (47) Serial beginning Jan. 29.
Form and Stability of Aeroplanes; researches Supplementing those of Lilienthal and Langley.* W. R. Turnbull. (From *Physical Review.*) (19) Jan. 30.
The Manufacture of High-Speed Steel.* O. M. Becker. (10) Feb.
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Lay Out of a Horizontal Return Tubular Boiler 18 Feet Long by 72 Inches Diameter.* R. R. Sturgeon. (94) Feb.
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Le Salon de l'Aéronautique et des Véhicules industriels (Paris, 24-30 décembre 1908).* G. Espitallier. (33) Jan. 9.
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Bericht über die Arbeiten der Kommission zur Ermittlung des Kraftbedarfs an Walzwerken.* H. Ortmann. (Paper read before the Verein Deutscher Eisenhüttenleute.) (50) Jan. 6.
Das Verzinnen von Metallgegenständen in Alter und Neuer Zeit. Otto Vogel. (Paper read before the Deutsche Giessereifachleute.) (50) Jan. 13.
Betrachtungen über das Kupolofenschmelzen mit Trockenem und Nassem Koks.* C. Geiger. (Paper read before the Deutsche Giessereifachleute.) (50) Jan. 13.
Elektrisch Betriebener Bockkran für die Verladung von Ingots.* R. Dub. (48) Jan. 16.
Feuerungskontrolle und Dampfkesseluntersuchungen.* Maercks. (48) Jan. 23.

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Tests on the Metallurgy of Steel being conducted at Watertown Arsenal, Mass. (89) Vol. 8.
Can Ingotism be Cured by Prolonged Exposure to the Temperature at which Overheating is Cured?* Henry M. Howe, William Campbell and W. T. Koken. (89) Vol. 8.
Some Practical Applications of Metallography.* William Campbell. (89) Vol. 8.
Report of the Sub-Committee on Metallography. (Amer. Soc. for Testing Materials.) (89) Vol. 8.
Some American Blast Furnace Plants.* (12) Serial beginning Jan. 8.
The Manufacture of Steel for Castings.* Bradley Stoughton. (Abstract of paper read before the Pittsburg Foundrymen's Assoc.) (47) Jan. 8.
Slag Car used at the Cananea Smelting Works.* Charles F. Shelby. (16) Jan. 23.
The Central Zinc Company's Works at Seaton Carew.* (11) Jan. 29.
The Mysteries of Metals.* J. O. Arnold. (Paper read before the Royal Inst.) (11) Jan. 29.
The Carrying Capacity of Launders; the Results of some Experiments with Launders of Glass, Wood, and Linoleum at different Grades.* W. C. Browning. (45) Feb.
Note sur les Couleurs de Revenu. Léon Guillet et Albert Portevin. (93) Jan.
Die Entzinnung der Weissblechabfälle und Ihre Wirtschaftliche Bedeutung. K. Goldschmidt. (Paper read before the Verein Deutscher Eisenhüttenleute.) (50) Dec. 30.
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Experimentelle Untersuchung des Thomasprozesses. (Abstract from *Métallurgie.*) (50) Jan. 27.

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*Illustrated.

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OF

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(INSTITUTED 1852)

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The House of the Society is open from 9 A.M. to 10 P.M. every day, except Sundays Fourth of July, Thanksgiving Day and Christmas Day.

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PROCEEDINGS

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MINUTES OF MEETINGS

OF THE SOCIETY

February 17th, 1909.—The meeting was called to order at 8.30 P. M.; George W. Tillson, Director Am. Soc. C. E., in the chair; Chas. Warren Hunt, Secretary; and present, also, 102 members and 6 guests.

A paper by R. P. Bolton, M. Am. Soc. C. E., entitled "The Operation of Passenger Elevators," was presented by the author and discussed orally by Messrs. R. B. Stanton, W. J. Boucher, F. Lavis, Noah Cummings, and the author.

Adjourned.

March 3d, 1909.—The meeting was called to order at 8.30 P. M.; President Onward Bates, in the chair; Chas. Warren Hunt, Secretary; and present, also, 252 members and 50 guests.

The minutes of the Annual Meeting, January 20th, and of the meeting of February 3d, 1909, were approved as printed in the *Proceedings* for February, 1909.

The President appointed Messrs. T. Kennard Thomson, Frank E. Winsor, and A. S. Nye as Tellers to canvass the ballot on the following proposed amendment to the Constitution:

Amend Article III as follows:

Section 1.—Amend the last paragraph to read as follows:

"All members other than Honorary Members and Juniors shall be admitted to the Society only by vote of the Corporate Members, as hereinafter specified."

Section 3.—Amend the last sentence of the first paragraph to read as follows:

"Not less than twenty days after the issue of such list, the Board of Direction shall consider these applications, together with any information in regard to the applicants that may have been received; may make further inquiries, if deemed expedient; shall classify the applicant, with his consent, and on applications for admission may direct a ballot."

Amend the last paragraph to read as follows:

"The Board shall have the power to elect persons to the grade of Junior, and to transfer persons from any grade except that of Junior to a higher grade of membership, and shall notify the Membership of its action."

Section 4.—Amend Section 4 to read as follows:

"The ballots shall be letter-ballots, in a form to be prescribed by the Board of Direction. They shall be mailed to each Corporate Member whose address is known, and shall state the date on which the ballot is to be canvassed, which shall be not less than twenty days after the issue of the ballot. **Negative votes equal to one per cent., or to the whole number nearest to one per cent., of the total corporate membership at the time of voting shall exclude from election.**

"A rejected applicant may renew his application for membership at any time after the expiration of one year from the date of the ballot rejecting his previous application."

The Tellers reported as follows:

On the adoption of the amendment to Article III, relating to the Method of Election of Members:

Total number of votes received		1 016
Not entitled to vote		3
Counted		1 013
No	762	
Yes	247	
Defective	4	
		1 013

An affirmative vote of two-thirds of all ballots cast being necessary for the adoption of an amendment to the Constitution, the President declared the amendment lost.

A paper by Messrs. Ernest R. Matthews and James Watson, entitled "The Action of Frost on Cement and Cement Mortar, Together With Other Experiments on These Materials," was presented by title and discussed orally by Messrs. J. L. Davis, R. B. Stanton, E. W. Stern, and John C. Wait; and the Secretary read communications on the subject from Messrs. William M. Venable and Herbert W. Hatton.

A paper by E. P. Goodrich, M. Am. Soc. C. E., entitled "The Bonding of New to Old Concrete," was presented by the author and discussed orally by Messrs. E. J. Fort, J. L. Davis, Myron H. Lewis, E. W. Stern, and the author. A communication on the subject from H. R. Burroughs, Jun. Am. Soc. C. E., was presented by the Secretary.

The Secretary announced the election of the following candidates by the Board of Direction on March 2d, 1909:

As Members.

Thomas Throp Allard, Havana, Cuba.
William Wallace Atterbury, Philadelphia, Pa.
Herschel Albert Benedict, Albany, N. Y.
Charles Slauson Boardman, Buffalo, N. Y.
Edward Webster Crellin, Des Moines, Iowa.
Louis Curtis Kelsey, Salt Lake City, Utah.
Guy McMurtry, Yuba City, Cal.
James Augustus Nelson, Pittsburg, Pa.
Samuel Henry Pitcher, Worcester, Mass.
Erdis Geroska Robinson, Columbus, Ohio.
Gilman Walter Smith, Chicago, Ill.

As Associate Members.

Robert Fleming Arnott, New York City.
John Charles Blaylock, Chicago, Ill.
Paul Bruce Breneman, State College, Pa.
Alexander Chisholm Copland, Richmond, Va.
Charles Wells Eddy, Thomaston, Conn.
Oscar Llewellyn Grover, Richmond, Va.
Benjamin Alexander Hodgdon, New York City.
Frank Rhymal Judd, Chicago, Ill.
Harlow Barton Kirkpatrick, Manila, Philippine Islands.
Morton Macartney, Spokane, Wash.
John Joseph Murphy, Yonkers, N. Y.
Thomas Jett Powell, Washington, D. C.
Eugene Jesse Rights, New York City.

As Associate.

Edward Michael Graves, Cleveland, Ohio.

As Juniors.

Harris Arkush Appel, Denver, Colo.
Franklin Babcock, Corregidor Island, Philippine Islands.
Harry Everett Barnes, New York City.
Fred Eberspacher, Vicksburg, Miss.
Harold Frederick Forsyth, Lone Tree, Wash.
Leon Cohen Heilbronner, Schenectady, N. Y.
Grover Cleveland Pruett, Miles City, Mont.
Paul Francis Rossell, Juana Diaz, Porto Rico.
Charles Willett Spooner, Ann Arbor, Mich.
Edwin Delevan Tillson, New York City.
Benjamin Franklin Vandervoort, New York City.
Louis Wachtel, Albany, N. Y.

The Secretary announced the transfer of the following candidates by the Board of Direction on March 2d, 1909:

From Associate Member to Member.

Clarence Edson Alderman, Boston, Mass.
Harry Franklin Bascom, Allentown, Pa.
Henry Baum Brewster, Syracuse, N. Y.
Charles Albert McKenney, Washington, D. C.
Eugene Everett Pettee, Boston, Mass.
Francis Lansing Pruyn, New York City.
Walter Evans Spear, Brooklyn, N. Y.
William MacIntire White, New York City.

From Junior to Associate Member.

Francis Fielding Longley, Washington, D. C.
Robert Faulkner Moss, Iloilo, Philippine Islands.
Alfred Marshall Wyman, East Orange N. J.

The Secretary announced the transfer of the following candidate by the Board of Direction on February 2d, 1909:

From Associate to Associate Member.

Richard Roswell Lyman, Salt Lake City, Utah.

The Secretary announced the election of the following candidate by the Board of Direction on September 1st, 1908:

As Junior.

Julius Lilien Jacobs, Houston, Tex.

The Secretary announced the following death:

Francis Smith Burrowes, elected Member June 6th, 1888; died February 15th, 1909.

Adjourned.

ANNOUNCEMENTS

The House of the Society is open from 9 A. M. to 10 P. M., every day, except Sundays, Fourth of July, Thanksgiving Day, and Christmas Day.

MEETINGS

Wednesday, April 7th, 1909.—8.30 P. M.—A paper on "The Sixth Street Viaduct of Kansas City," by E. E. Howard, Assoc. M. Am. Soc. C. E., will be presented for discussion.

This paper was printed in *Proceedings* for February, 1909.

Wednesday, April 21st, 1909.—8.30 P. M.—Two papers will be presented for discussion, as follows: "The Maximum Weights of Slow Freight Trains," by C. S. Bissell, M. Am. Soc. C. E.; and "Sampittic Surfacing," by W. W. Crosby, M. Am. Soc. C. E.

These papers were printed in *Proceedings* for February, 1909.

Wednesday, May 5th, 1909.—8.30 P. M.—Two papers will be presented for discussion, as follows: "A System of Cost Keeping," by Myron S. Falk, Assoc. M. Am. Soc. C. E.; and "The Design of Elevated Tanks and Stand-Pipes," by C. W. Birch-Nord, Assoc. M. Am. Soc. C. E..

These papers are printed in this number of *Proceedings*.

Wednesday, May 19th, 1909.—8.30 P. M.—A paper entitled "The Sewer System of San Francisco, and a Solution of the Storm-Water Flow Problem," by C. E. Grunsky, M. Am. Soc. C. E., will be presented for discussion.

This paper is printed in this number of *Proceedings*.

ANNUAL CONVENTION

The Forty-first Annual Convention of the Society will be held at the Mount Washington Hotel, Bretton Woods, N. H., from July 6th to July 9th, 1909, inclusive.

The general arrangements for the Convention are in the hands of the following Committees:

COMMITTEE OF THE BOARD OF DIRECTION

F. W. Hodgdon,

G. W. Tillson, Chas. Warren Hunt.

LOCAL COMMITTEE

H. W. Hayes,

A. W. Dean, S. E. Tinkham,
H. D. Woods, J. F. Stevens,
George A. Kimball, J. W. Ellis.

PAPERS AND DISCUSSIONS

The first volume of *Transactions* for 1909 (Vol. LXII) will be issued in about ten days. There will be three additional volumes issued for this year. As will be seen, this volume is as large as those recently issued semi-annually.

It is hoped that members and others who take part in the discussion of the papers presented will revise their remarks promptly, and that all written communications from those who cannot attend the meetings will be sent in at the earliest possible date after the issue of a paper in *Proceedings*. The issue of volumes of *Transactions* is dependent on the closing of discussions, and the co-operation of the membership will now be more necessary in this matter than heretofore, because four volumes are to be issued during the year instead of two, and, to accomplish this, a definite date of issue for each has been established. It is expected that the second volume for 1909 will be issued on June 30th, and the third and fourth on September 30th and December 31st, respectively.

All papers accepted by the Publication Committee are classified by the Committee with respect to their availability for discussion at meetings.

Papers, which, from their general nature, appear to be of a character suitable for oral discussion will be published as heretofore in *Proceedings,* and set down for presentation to a future meeting of the Society, and, on these, oral discussion, as well as written communications, will be solicited.

All papers which do not come under this heading, that is to say, those which, from their mathematical or technical nature, in the opinion of the Committee, are not adapted to oral discussion, will not be scheduled for presentation to any meeting. Such papers will be published in *Proceedings* in the same manner as those which are to be presented at meetings, but written discussions, only, will be requested for subsequent publication in *Proceedings* and with the paper in the volumes of *Transactions.*

PRIVILEGES OF ENGINEERING SOCIETIES EXTENDED TO MEMBERS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

Members of the American Society of Civil Engineers will be welcomed by the following Engineering Societies, both to the use of their Reading Rooms and at all Meetings:

American Institute of Mining Engineers, 29 West Thirty-ninth Street, New York City.

Associação dos Engenheiros Civis Portuguezes, Lisbon, Portugal.

Australasian Institute of Mining Engineers, Melbourne, Victoria, Australia.

Boston Society of Civil Engineers, 715 Tremont Temple, Boston, Mass.

Brooklyn Engineers' Club, 197 Montague Street, Brooklyn, N. Y.

Canadian Society of Civil Engineers, 877 Dorchester Street, Montreal, Que., Canada.

Civil Engineers' Club of Cleveland, 718 Caxton Building, Cleveland, Ohio.

Civil Engineers' Society of St. Paul, St. Paul, Minn.

Cleveland Institute of Engineers, Middlesbrough, England.

Colorado Association of Members, Am. Soc. C. E., 235 Equitable Building, Denver, Colo.

Engineers' and Architects' Club of Louisville, Ky., 303 Norton Building, Fourth and Jefferson Streets, Louisville, Ky.

Engineers' Club of Baltimore, Baltimore, Md.

Engineers' Club of Central Pennsylvania, Corner Second and Walnut Streets, Harrisburg, Pa.

Engineers' Club of Philadelphia, 1317 Spruce Street, Philadelphia, Pa.

Engineers' Club of St. Louis, 3817 Olive Street, St. Louis, Mo.

Engineers' Club of Toronto, 96 King Street, West, Toronto, Ont., Canada.

Engineers' Society of Western Pennsylvania, 803 Fulton Building, Pittsburg, Pa.

Institute of Marine Engineers, 58 Romford Road, Stratford, London, E., England.

Institution of Engineers of the River Plate, Buenos Aires, Argentine Republic.

Institution of Naval Architects, 5 Adelphi Terrace, London, W. C., England.

Junior Institution of Engineers, 39 Victoria Street, Westminster, S. W., London, England.

Koninklijk Instituut van Ingenieurs, The Hague, The Netherlands.

Louisiana Engineering Society, 604 Tulane-Newcomb Building, New Orleans, La.

Memphis Engineering Society, Memphis, Tenn.

Midland Institute of Mining, Civil and Mechanical Engineers, Sheffield, England.

Montana Society of Engineers, Butte, Montana.

North of England Institute of Mining and Mechanical Engineers, Newcastle-upon-Tyne, England.

Oesterreichischer Ingenieur- und Architekten-Verein, Eschenbachgasse 9, Vienna, Austria.

Pacific Northwest Society of Engineers, 617-618 Pioneer Building, Seattle, Wash.

Rochester Engineering Society, Rochester, N. Y.

Sachsischer Ingenieur- und Architekten-Verein, Dresden, Germany.

Sociedad Colombiana de Ingenieros, Bogota, Colombia.

Societe des Ingenieurs Civils de France, 19 Rue Blanche, Paris, France.

Society of Engineers, 17 Victoria Street, Westminster, S. W., England.

Svenska Teknologföreningen, Brunkebergstorg 18, Stockholm, Sweden.

Tekniske Forening, Vestre Boulevard 18–1, Copenhagen, Denmark.

Western Society of Engineers, 1737 Monadnock Block, Chicago, Ill.

SEARCHES IN THE LIBRARY

In January, 1902, the Secretary was authorized to make searches in the Library, upon request, and to charge therefor the actual cost to the Society for the extra work required. Since that time many searches have been made, and bibliographies and other information on special subjects furnished.

The resulting satisfaction, to the members, who have made use of the resources of the Society in this manner, has been expressed frequently, and leaves little doubt that, if it were generally known to the membership that such work would be undertaken, many would avail themselves of it.

The cost is trifling compared with the value of the time of an engineer who looks up such matters himself, and the work can be performed quite as well, and much more quickly, by persons familiar with the Library.

In asking that such work be undertaken, members should specify clearly the subject to be covered, and whether references to general books only are desired, or whether a complete bibliography, involving search through periodical literature, is desired.

In reference to this work the Appendix* to the Annual Report of the Board of Direction for the year ending December 31st, 1906, contains a summary of all searches made to that date.

* *Proceedings.* Vol. XXXIII, p. 20 (January, 1907).

LOCAL ASSOCIATIONS OF MEMBERS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

San Francisco Association

(Abstract of Minutes of Meeting)

December 16th, 1908.—The annual meeting of the San Francisco Association of Members, Am. Soc. C. E., was held; President C. D. Marx in the chair; Franklin Riffle, Secretary; and present, also, 38 members and guests.

The minutes of the previous meeting were approved as printed in the *Proceedings* of the Association.

The Secretary presented his annual report. Since the last annual report, six meetings have been held, the average attendance being 38. There have been 8 additions to the membership, and 5 names have been dropped, making the present membership 102, a net gain of 3.

The Treasurer reported that the receipts for the year had been $1 505.49, and the disbursements $549.65, leaving a balance of $955.84.

The Secretary read a letter from Charles Warren Hunt, Secretary of the American Society of Civil Engineers, conveying to the Association (as suggested by the Committee to Recommend the Award of Prizes) the appreciation of the Board of Direction of the Society for the work done by the Association in preparing papers pertaining to the "Effects of the San Francisco Earthquake of April, 1906."

The Secretary was instructed to convey to the Secretary of the Society the thanks of the Association for the words of commendation contained in his letter.

The following officers were elected:

President, LUTHER WAGONER.
Vice-Presidents, J. D. GALLOWAY,
C. B. WING.
Secretary, E. T. THURSTON, JR.
Treasurer, P. E. HARROUN.

Mr. L. J. Mensch gave an interesting talk concerning the manufacture and driving of 190 reinforced concrete piles for the power station of the Pacific Gas and Electric Company, in Oakland, Cal.

Adjourned.

Colorado Association

The Colorado Association of Members of the American Society of Civil Engineers was organized at a meeting held in Denver, Colo., in December, 1908. A constitution, which has subsequently been approved

by the Board of Direction of the Society, was adopted February 13th, 1909, and the following officers were elected:

President, HERBERT S. CROCKER.
Vice-President, GEORGE G. ANDERSON.
Secretary-Treasurer, HENRY J. BURT.

Meetings are held on the second Saturday of each month except July and August.

THE JOHN FRITZ MEDAL

The John Fritz Medal for 1909 has been awarded to Mr. Charles T. Porter for his work in "advancing the knowledge of steam engineering and in improvements in engine construction."

It has been decided that the presentation of this medal shall be made at a public meeting to be held in the auditorium of the Engineering Societies Building, 29 West 39th Street, on the evening of Tuesday, April 13th, 1909.

Addresses will be made on the following subjects:

"The Debt of Modern Industrial Civilization to the Steam Engine,"
"The Debt of the Steel Rolling Mill to the High-Speed Engine,"
"The Debt of the Electrical Generator to the High-Speed Engine,"
"The Debt of the Modern Steam Engine to Charles T. Porter."

All members of this Society are invited to attend this meeting.

ACCESSIONS TO THE LIBRARY

(From February 10th to March 9th, 1909)

DONATIONS.*

THEORY AND DESIGN OF REINFORCED CONCRETE ARCHES.

A Treatise for Engineers and Technical Students. By Arvid Reuterdahl. Cloth, 9 x 6 in., illus., 3 + 126 pp. Chicago and New York, The Myron C. Clark Publishing Co., 1908. $2.00 net.

The claim of the author of this work, as set forth in his preface, is that there has been no previous treatise on the analysis of the elastic arch which was not either so mathematically abstruse, or left so much to the reader to demonstrate for himself, as to be of little use to the practical engineer or technical student whose mathematical training has not been of an exceptional order. He further claims that every possible principle involved in the graphical treatment of analysis is explained thoroughly in the theoretical portion of his work, and that there are no missing steps in the necessary mathematical analysis of the theory, as set forth in the treatise. The Contents are: Theory of the Elastic Arch; Design of a Reinforced Concrete Arch; Calculations of Fiber Stresses; Index.

ASPHALTS: THEIR SOURCES AND UTILIZATIONS.

Asphalts for Dustless Roads; Recent Improvements in Asphalt Industries, Together with Addenda Treating on General Waterproof Construction. By T. Hugh Boorman. Cloth, 10 x 6½ in., illus., 168 pp. New York, William T. Comstock, 1908. $3.00.

The author, in his preface, states that he has endeavored to furnish a complete manual on asphalt, which shall be a reliable reference book. The book was suggested, he says, by a series of articles that appeared in *Architects' and Builders' Magazine*, for which, since the numbers containing them ran out of print, there has been a continuous call, and the request was made that they be issued in book form. The Contents are: Discovery and Early Use of Asphalt; Rock Asphalt Mastic or Asphalt Coulé; Trinidad Asphalt; Petroleum Residuum and California Malthas as a Fluxing Material; Venezuela Asphalts; Cuban Asphalts; American Bituminous Limestone; Bituminous Asphalt Sandstone Rock; Manjak and Uintaite; Late European Work; Turkish and Other Bitumens; Developments of Asphalt Industry up to 1903; Asphalts in 1908; Asphalt in Building Construction; Dustless Roads; Methods of Surfacing Roads; Asphaltic Oils, Their Classification and Properties; Application of Asphaltic Oils; Sprinkling with Asphaltic Oils; Latest Views of Engineers on Asphaltic Surfacing; Municipal Asphalt Plants; Asphalt Waterproofing; Asphalt in Roofing; Asphalt in Manufacture; Asphalt Machinery; Index.

ENGINEERS' POCKETBOOK OF REINFORCED CONCRETE.

By E. Lee Heidenreich. Roan, 7 x 4½ in., illus., 9 + 364 pp. Chicago and New York, The Myron C. Clark Publishing Co.; London, E. & F. Spon, Limited, 1909. $3.00 net.

The preface states that the author has been writing, changing, and improving this book for over eight years, and that, owing to the comparative newness of the subject and its possibilities for additions and amendments, it has been almost impossible to find a place at which it could be considered even temporarily finished. A pocketbook on the subject being much needed, however, the author here gives, more or less concisely, the result of about fifteen years of study and experience in exploitation and construction of reinforced concrete. The Contents are: Materials and Machines Used in Reinforced Concrete Construction; Design and Construction of Buildings; The Design and Construction of Bridges; Abutments and Retaining Walls; Culverts, Conduits, Sewers, Pipes, and Dams; Tanks, Reservoirs, Bins, and Grain Elevators; Chimneys, Miscellaneous Data, Cost Keeping, Estimating, Specifications, Etc.; Index.

*Unless otherwise specified, books in this list have been donated by the publisher.

GENERAL LECTURES ON ELECTRICAL ENGINEERING.

By Charles Proteus Steinmetz. Edited by Joseph Le Roy Hayden. Cloth, 9 x 6 in., illus., 284 pp. Schenectady, N. Y., Robson & Adee. $2.00 net.

This book, the publishers announce, is made up of a series of lectures, dealing with the operation of electric systems and apparatus under normal and abnormal conditions, and with their design. The author's preface states, however, that the design of apparatus has been discussed only so far as it is necessary to understand their operation and so judge of their proper field of application, and that the lectures comprise a discussion of the different methods of application of electric energy, the means and apparatus available, the different methods of carrying out the purpose, and the relative advantages and disadvantages of the different methods and apparatus which determine their choice, the treatment being essentially descriptive and not mathematical. The Lecture Headings are: General Review; General Distribution; Light and Power Distribution; Load Factor and Cost of Power; Long Distance Transmission; Higher Harmonics of the Generator Wave; High Frequency Oscillations and Surges; Generation; Hunting of Synchronous Machines; Regulation and Control; Lightning Protection; Electric Railway, Train Characteristics; Electric Railway, Motor Characteristics; Alternating Current Railway Motor; Electro-chemistry; The Incandescent Lamp; Arc Lighting; Appendix I, Light and Illumination; Appendix II, Lightning and Lightning Protection.

AUDEL'S GAS ENGINE MANUAL.

A Practical Treatise Relating to the Theory and Management of Gas, Gasoline, and Oil Engines, Including Chapters on Producer Gas Plants, Marine Motors, and Automobile Engines. Cloth, 8½ x 5½ in., illus., 24 + 469 pp. New York, Theo. Audel & Co. $2.00.

The Contents are: Historical Development; Laws of Permanent Gases; Theoretical Working Principles; Actual Working Cycles; Graphics of the Action of Gases; Indicator Diagrams of Engine Cycles; Indicator Diagrams of Gas Engines; Fuels and Explosive Mixtures; Gas Producer Systems; Compression, Ignition and Combustion; Design and Construction; Governing and Governors; Ignition and Igniters; Installation and Operation; Four-Cycle Horizontal Engines; Four-Cycle Vertical Engines; Four-Cycle Double-Acting Engines; Two-Cycle Engines; Foreign Engines; Oil Engines; Marine Engines; Testing; Instruments Used in Testing; Nature and Use of Lubricants; Hints on Management and Suggestions for Emergencies; The Automobile; Useful Rules and Tables; Index.

PATENTS AS A FACTOR IN MANUFACTURING.

By Edwin J. Prindle. Cloth, 7¾ x 5 in., 134 pp. New York, The Engineering Magazine, 1908. $2.00.

The author of this book disclaims any purpose to make the inventor or manufacturer his own patent attorney, and the editor's preface states it to be designed especially to lay down the fundamental principles, so that they may be grasped clearly and fully enough to direct the course of the inventor, patentee, or manufacturer, in the early steps which are usually taken before the advice of counsel is secured. The Contents are: Influence of Patents in Controlling a Market; Subject, Nature, and Claim of a Patent; What Protection a Patent Affords; Of Infringements; Patenting a New Product; Patent Relations of Employer and Employee; Contests Between Rival Claimants to an Invention; Index.

PRACTICAL CALCULATION OF TRANSMISSION LINES

For Distribution of Direct and Alternating Currents by Means of Overhead, Underground, and Interior Wires for Purposes of Light, Power, and Traction. By L. W. Rosenthal. Cloth, 9½ x 6 in., 10 + 93 pp. New York, McGraw Publishing Company, 1909. $2.00 net.

The chief mission of this book is stated by the author to be to substitute a direct solution for the trial method of calculation of transmission lines which was formerly a necessary evil. The scope of the book has been confined to methods of calculation, it is stated, and the arrangement of the formulas, tables, and text dictated solely by the needs of the rapid worker. All sections, except the last, include the effects of temperature and specific conductivity. The section relating to direct-current railways is claimed to be novel in the form of its tables, and the methods outlined in it, rapid and comprehensive; that relating to alternating-

current transmission is said to present an original method for the solution of these problems. The Chapter Headings are: Direct-Current Distribution for Light and Power; Distribution for Direct-Current Railways; Alternating-Current Transmission by Overhead Wires; Alternating-Current Transmission by Underground Cables; Interior Wires for Alternating-Current Distribution; Distribution for Single-Phase Railways.

HEAT ENERGY AND FUELS.

Pyrometry, Combustion, Analysis of Fuels, and Manufacture of Charcoal, Coke, and Fuel Gases. By Hanns v. Jüptner. Translated by Oskar Nagel. Cloth, 9½ x 6 in., illus., 5 + 306 pp. New York, McGraw Publishing Company, 1908. $3.00 net.

The translator is authority for the statement that this volume contains a large amount of carefully tabulated data, of which a great deal is new, in convenient form for use, and that while the book is intended for use in universities and engineering schools, it is of equal value to practising engineers, since it gives, not only the fundamental principles, but also the latest experimental data and practice. The Contents are: General Remarks; Forms of Energy; The Measurement of High Temperatures (Pyrometry); Pyrometry (continued); Pyrometry (conclusion); Optical Methods of Measuring Temperatures; Combustion Heat and Its Determination; Direct Methods for Determining the Combustion Heat; Incomplete Combustion; Combustion Temperature; Fuels (In General); Wood; Fossil Solid Fuels (In General); Peat; Brown Coal (Lignite); Bituminous and Anthracite Coals; Artificial Solid Fuels; Charcoal; Peat-Coal, Coke, and Briquettes; Coking Apparatus; Liquid Fuels; Gaseous Fuels; Producer Gas; Water Gas; Dowson Gas, Blast Furnace Gas, and Regenerated Combustion Gases; Apparatus for the Production of Fuel Gases; Index.

LAW AND BUSINESS OF ENGINEERING AND CONTRACTING.

With Numerous Forms and Blanks for Practical Use. By Charles Evan Fowler, M. Am. Soc. C. E. Cloth, 9 x 6 in., 9 + 162 pp. New York, McGraw Publishing Company, 1909. $2.50 net.

The author states in his preface that this book is the elaboration of a series of lectures delivered to the engineering students of Washington University, with some additional matter in the shape of forms of contracts, specifications, and blank business forms. The Contents are: The Relation Between the Engineer and Contractor; Ordinary Forms of Contracts; Ordinary Specifications; Special Forms of Specifications; Special Forms of Contracts; Inspection of Engineering Work; Estimating Materials and the Cost of Engineering Structures; Bidding on Engineering Work; Organization of Contract Work; Essentials of Contract Law; Index.

THE OCEAN CARRIER.

A History and Analysis of the Service, and a Discussion of the Rates of Ocean Transportation. By J. Russell Smith. Cloth, 7¾ x 5 in., illus., 11 + 344 pp. New York and London, G. P. Putnam's Sons, 1908. $1.50.

The author calls this book primarily an economic study of the ocean service, and states it to be the outgrowth of the study of three questions—the development of line traffic, the combinations among carriers to control rates, and the combination of steamship lines and railways. It is written in a popular, rather than a technical style, and treats of what ships carry, where they carry it, for whom, at what rate, under what management, etc., tracing the main lines of past development and the dominant factors in the present situation. The Contents are: Part I, The Service of the Ocean Carrier: Ship Development—the Evolution of the Vehicle; The Organization of Ocean Carrying; The Leading Routes of Ocean Commerce; The Epoch of the Merchant Carrier on the Sea; Line Traffic and Its Extension; Recent Developments in Line Traffic; The Normal Type of Steamship Line Organization; The Railroad Steamship Lines on the Atlantic and Gulf Coasts of the United States; The Railroad Steamship Lines on the Pacific Coast of the United States and in Europe; The Renaissance of the Merchant Carrier—The Private Steamship Line; Line Traffic in the United States Coasting Trade. Part II, The Rates of the Ocean Carrier: Charter Freight Rates and Attempts at Their Control; Factors Affecting Ocean Line Freight Rates; Agreements Among Ocean Carriers to Control Rates; Transatlantic Freight Rates and Their Control—The Lines Between America and Great Britain; Transatlantic Freight Rates and Their Control—The Lines Between America and the Continent; The Present Situation and Future Outlook; Index.

LABORATORY EXPERIMENTS IN METALLURGY.

By Albert Sauveur and H. M. Boylston. Cloth, 10¼ x 7¾ in., illus., 73 pp. Cambridge, Mass., The Authors, 1908. $1.25. (Presented by the Authors.)

This book is made up of short notes written primarily for the use of students at Harvard University, who take the courses in General Metallurgy and in Metallurgy of Iron and Steel. The authors' intention, as indicated in the preface, is not to deal with the subject exhaustively, but to make the notes suggestive to other teachers and students in metallurgy, and even to practitioners of the art. The book is divided into two parts of thirteen experiments each, Part I dealing with experiments in general metallurgy, and Part II with experiments in metallurgy of iron and steel. There are various tables in the text such as Typical Analyses of Various Fuels, Melting Points of Metals, etc., and illustrations of apparatus. After each experiment a blank laboratory report is given to be filled out by the student.

Gifts have also been received from the following:

Alexandra (Newport & South Wales) Docks & Ry. Co. 1 pam.
Allied Real Estate Interests of the State of New York. 1 pam.
Am. Assoc. for the Advancement of Sci. 1 vol.
Am. Inst. of Min. Engrs. 1 bound vol.
Am. Ry. Bridge and Bldg. Assoc. 1 vol.
Am. Soc. of Heating and Ventilating Engrs. 1 bound vol.
Am. Soc. of Mech. Engrs. 1 bound vol.
Baker Street & Waterloo Ry. Co. 1 pam.
Belzner, Theodore. 2 pam.
Bengal, India-Irrig. Dept. 1 bound vol.
Binghamton, N. Y.-Water Commrs. 1 pam.
Bombay-Public Works Dept. 1 pam.
Boston, Mass.-Transit Comm. 1 bound vol.
Brockton, Mass.-Water Commrs. 1 pam.
Burlington, Vt.-City Clerk. 1 pam.
California-State Engr. 1 vol.
Cambrian Rys. Co. 1 pam.
Carnegie Library of Pittsburgh. 1 pam.
Case School of Applied Science. 1 pam.
Charing Cross, Euston & Hampstead Ry. Co. 1 pam.
Chicago Terminal Transfer R. R. Co. 1 pam.
Cleveland Eng. Soc. 1 pam.
Compagnie du Chemin de Fer du Congo. 1 pam.
Cooper, Theodore. 1 vol.
Cornell Univ. Library. 1 bound vol., 1 pam.
Delaware, Lackawanna & Western R. R. Co. 1 pam.
Deutsch-Amerikanischer Techniker-Verband. 1 pam.
Eng. Assoc. of New South Wales. 1 bound vol.
Engrs.' Club of Philadelphia. 1 pam.
Engrs.' Club of Toronto. 1 bound vol.
Furness Ry. Co. 1 pam.
Gloucester, Mass.-Water Commrs. 1 pam.
Great Central Ry. Co. 2 pam.
Great Northern, Piccadilly & Brompton Ry. Co. 1 pam.
Great Western Ry. Co. 1 pam.
Harrisburg, Pa.-Water and Light Dept. 1 pam.
Haverhill, Mass.-Board of Water Commrs. 1 pam.
Hollyday, R. C. 1 pam.
Hull & Barnsley Ry. Co. 1 pam.
India-Ry. Board. 1 pam.
Institution of Gas Engrs. 1 bound vol.
Great Northern Ry. Co. (Ireland). 1 pam.
Iron and Steel Inst. 1 bound vol.
Junior Inst. of Engrs. 1 bound vol.
Lakes-to-the-Gulf Deep Waterway Assoc. 2 pam.
Lee, F. V. T. 1 pam.
Levy, Ernest C. 1 pam.
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BY PURCHASE.

Coal. By James Tonge. New York, D. Van Nostrand Co., 1907.

The Mechanical Engineering of Steam Power Plants. By F. R. Hutton. Ed. 3, Rewritten. New York, John Wiley & Sons; London, Chapman & Hall, Limited, 1909.

The Temperature-Entropy Diagram. By C. W. Berry. Ed. 2, Rev. and Enl. New York, John Wiley & Sons; London, Chapman & Hall, Limited, 1908.

Die Eisernen Brücken im Allgemeinen: Theorie der Eisernen Balkenbrücken. Handbuch der Ingenieur Wissenschaften. II. Teil, dritter Band, vierte vermehrte Auflage. Bearbeitet von J. E. Brik, Th. Landsberg und Fr. Steiner. Leipzig, Wilhelm Engelmann, 1909.

Physical Geography of the Texas Region. By R. T. Hill. Topographic Atlas of the United States. Washington, United States Geological Survey, 1900.

Steam-Boilers. By C. H. Peabody and E. F. Miller. Ed. 2, Rev. and Enl. New York, John Wiley & Sons; London, Chapman & Hall, 1908.

SUMMARY OF ACCESSIONS

(From February 10th to March 8th, 1909)

Donations (including 18 duplicates)	166
By purchase	6
Total	172

MEMBERSHIP

ADDITIONS

(February 10th to March 9th, 1909)

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WARD, GEORGE MERRITT. Gen. Mgr. of Constr., Suburban Constr. Co., 111 Broadway, New York City.

RESIGNATION

ASSOCIATE MEMBER	Date of Resignation
KINSEY, FRANK WILMARTH	Mar. 2, 1909

DEATH

BURROWES, FRANCIS SMITH. Elected Member, June 6th, 1888; died February 15th, 1909.

MONTHLY LIST OF RECENT ENGINEERING ARTICLES OF INTEREST

(February 9th to March 8th, 1909.)

NOTE.—*This list is published for the purpose of placing before the members of the Society, the titles of current engineering articles, which can be referred to in any available engineering library, or can be procured by addressing the publication directly, the address and price being given wherever possible.*

LIST OF PUBLICATIONS

In the subjoined list of articles, references are given by the number prefixed to each journal in this list:

(1) *Journal,* Assoc. Eng. Soc., 31 Milk St., Boston, Mass., 30c.
(2) *Proceedings,* Engrs. Club of Phila., 1317 Spruce St., Philadelphia, Pa.
(3) *Journal,* Franklin Inst., Philadelphia, Pa., 50c.
(4) *Journal,* Western Soc. of Engrs., Monadnock Blk., Chicago, Ill.
(5) *Transactions,* Can. Soc. C. E., Montreal, Que., Canada.
(6) *School of Mines Quarterly,* Columbia Univ., New York City, 50c.
(7) *Technology Quarterly,* Mass. Inst. Tech., Boston, Mass., 75c.
(8) *Stevens Institute Indicator,* Stevens Inst., Hoboken, N. J., 50c.
(9) *Engineering Magazine,* New York City, 25c.
(10) *Cassier's Magazine,* New York City, 25c.
(11) *Engineering* (London), W. H. Wiley, New York City, 25c.
(12) *The Engineer* (London), International News Co., New York City, 35c.
(13) *Engineering News,* New York City, 15c.
(14) *The Engineering Record,* New York City, 12c.
(15) *Railroad Age Gazette,* New York City, 15c.
(16) *Engineering and Mining Journal,* New York City, 15c.
(17) *Electric Railway Journal,* New York City, 10c.
(18) *Railway and Engineering Review,* Chicago, Ill., 10c.
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(23) *Bulletin,* American Iron and Steel Assoc., Philadelphia, Pa.
(24) *American Gas Light Journal,* New York City, 10c.
(25) *American Engineer,* New York City, 20c.
(26) *Electrical Review,* London, England.
(27) *Electrical World,* New York City, 10c.
(28) *Journal,* New England Water-Works Assoc., Boston, Mass., $1.
(29) *Journal,* Royal Society of Arts, London, England, 15c.
(30) *Annales des Travaux Publics de Belgique,* Brussels, Belgium.
(31) *Annales* de l'Assoc. des Ing. Sortis des Ecoles Spéciales de Gand. Brussels, Belgium.
(32) *Mémoires et Compte Rendu des Travaux,* Soc. Ing. Civ. de France, Paris, France.
(33) *Le Génie Civil,* Paris, France.
(34) *Portefeuille Economiques des Machines,* Paris, France.
(35) *Nouvelles Annales de la Construction,* Paris, France.
(37) *Revue de Mécanique,* Paris, France.
(38) *Revue Générale des Chemins de Fer et des Tramways,* Paris, France.
(41) *Modern Machinery,* Chicago, Ill., 10c.
(42) *Proceedings,* Am. Inst. Elec. Engrs., New York City, 50c.
(43) *Annales des Ponts et Chaussées,* Paris, France.
(44) *Journal,* Military Service Institution, Governors Island, New York Harbor, 50c.
(45) *Mines and Minerals,* Scranton, Pa., 20c.
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(49) *Zeitschrift für Bauwesen,* Berlin, Germany.
(50) *Stahl und Eisen,* Düsseldorf, Germany.
(51) *Deutsche Bauzeitung,* Berlin, Germany.
(52) *Rigasche Industrie-Zeitung,* Riga, Russia.
(53) *Zeitschrift,* Oesterreichischer Ingenieur und Architekten Verein, Vienna, Austria.
(54) *Transactions,* Am. Soc. C. E., New York City, $5.
(55) *Transactions,* Am. Soc. M. E., New York City, $10.
(56) *Transactions,* Am. Inst. Min. Engrs., New York City, $5.

(57) *Colliery Guardian*, London, England.
(58) *Proceedings*, Eng. Soc. W. Pa., 803 Fulton Bldg., Pittsburg, Pa., 50c.
(59) *Transactions*, Mining Inst. of Scotland, London and Newcastle-upon-Tyne, England.
(60) *Municipal Engineering*, Indianapolis, Ind., 25c.
(61) *Proceedings*, Western Railway Club, 225 Dearborn St., Chicago, Ill., 25c.
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(63) *Minutes of Proceedings*, Inst. C. E., London, England.
(64) *Power*, New York City, 20c.
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(76) *Brick*, Chicago, Ill., 10c.
(77) *Journal*, Inst. Elec. Engrs., London, England.
(78) *Beton und Eisen*, Vienna, Austria.
(79) *Forscherarbeiten*, Vienna, Austria.
(80) *Tonindustrie Zeitung*, Berlin, Germany.
(81) *Zeitschrift für Architektur und Ingenieurwesen*, Wiesbaden, Germany.
(82) *Dinglers Polytechnisches Journal*, Berlin, Germany.
(83) *Progressive Age*, New York City, 15c.
(84) *Le Ciment*, Paris, France.
(85) *Proceedings*, Am. Ry. Eng. and M. of W. Assoc., Chicago, Ill.
(86) *Engineering-Contracting*, Chicago, Ill., 10c.
(87) *Roadmaster and Foreman*, Chicago, Ill., 10c.
(88) *Bulletin* of the International Ry. Congress Assoc., Brussels, Belgium.
(89) *Proceedings*, Am. Soc. for Testing Materials, Philadelphia, Pa.
(90) *Transactions*, Inst. of Naval Archts., London, England.
(91) *Transactions*, Soc. Naval Archts. and Marine Engrs., New York City.
(92) *Bulletin*, Soc. d'Encouragement pour l'Industrie Nationale, Paris, France.
(93) *Revue de Métallurgie*, Paris, France, 4 fr. 50.
(94) *The Boiler Maker*, New York City, 10c.
(95) *International Marine Engineering*, New York City, 20c.

LIST OF ARTICLES.

Bridge.

The Detailing of Skew Portals.* J. P. Davies. (13) Feb. 11.
Modern Methods of Bridge Construction. (12) Feb. 19.
The Design, Construction and Cost of a Reinforced Concrete Trestle.* (14) Feb. 20.
Concrete Work on Sparkman St. Bridge, Nashville, Tenn.* W. F. Creighton. (13) Feb. 25.
Note sur le Rétablissement de la Circulation sur la Rive Droite du Rhône à la Suite des Coupures par les Inondations d'Octobre 1907.* Ruffieux. (38) Feb.
Pont Suspendu Fixé (Système Gisclard) de la Cassagne (Pyrénées-Orientales).* G. Leinekugel Le Cocq. (33) Serial beginning Feb. 20.
Ueber den Festen Anschluss der Querträger an die Hauptträger. P. Müller. (81) Vol. 6, 1908.
Die Auswechselung der Humboldthafen-Brücken in Berlin.* C. Müller. (48) Jan. 30.

Electrical.

Some Results of Experience with Electrically Driven Rolling-Mills.* C. Koettgen, M. Inst. C. E., and C. A. Ablett, A. M. Inst. C. E. (71) Vol. 78.
Power Supply and Its Effect on the Industries of the North-East Coast.* Charles H. Merz. (71) Vol. 78.
Representative Data from Electric Power-Plant Operation. Howard S. Knowlton. (9) Feb.
The Urft Valley Energy Transmission Plant.* (27) Feb. 11.
Electricity Supply in Bristol, 1909.* (26) Serial beginning Feb. 12.
Electric Power Installation at Khartoum.* (11) Feb. 12.
Some Motor Installations on the Mains of the Lancashire Electric Power Co.* (73) Feb. 12.
The Use of Large Gas Engines for Generating Electric Power.* Leonard Andrews and R. Porter. (Abstract of paper read before the Inst. of Elec. Engrs.) (73) Serial beginning Feb. 12; (47) Serial beginning Feb. 19; (12) Feb. 19.
Inductance Coils Used in Wireless Telegraphy.* John L. Hogan, Jr. (27) Feb. 18.

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*Illustrated.

Electrical—(Continued).

Transformer Substation for Niagara Energy at Rochester, N. Y.* (27) Feb. 18.
Water-Power and Transformer Stations of the Rochester Railway & Light Company.* (17) Feb. 20.
Electrical Equipment of the Bergenport Chemical Works.* Warren H. Miller. (27) Feb. 25.
Flywheel Load Equalizer.* J. S. Peck. (Paper read before the Inst. of Elec. Engrs.) (73) Feb. 26; (22) Feb. 26.
The Industrial Application of the Electric Motor as Illustrated in the Gary Plant of the Indiana Steel Company.* B. R. Shover. (42) Mar.
Condenser Type of Insulation for High-Tension Terminals.* A. B. Reynders. (42) Mar.
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*Illustrated.

PROCEEDINGS

OF THE

AMERICAN SOCIETY

OF

CIVIL ENGINEERS

(INSTITUTED 1852)

VOL. XXXV—No. 4

APRIL, 1909

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NEW YORK 1909

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The House of the Society is open from 9 A.M. to 10 P.M. every day, except Sundays, Fourth of July, Thanksgiving Day and Christmas Day.

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AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PROCEEDINGS

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MINUTES OF MEETINGS

OF THE SOCIETY

March 17th, 1909.—The meeting was called to order at 8.30 P. M.; Vice-President George H. Pegram in the chair; Chas. Warren Hunt, Secretary; and present, also, 155 members, and 42 guests.

A paper by L. R. Gifford, Assoc. M. Am. Soc. C. E., entitled "Steel Sheeting and Sheet-Piling," was presented by the author.

Communications on the subject from Messrs. Charles W. Sherman and C. C. Conkling were read by the Secretary; and the paper was discussed orally by Messrs. E. P. Goodrich, F. W. Skinner, J. C. Meem, R. B. Woodworth, and the author.

The Secretary anounced the following deaths:

HOWARD BREEN, elected Member April 4th, 1888; **died February 14th, 1909.**

CHARLES HERBERT DEANS, elected Junior December 3d, 1890; Associate Member May 6th, 1896; died March 7th, 1909.

Adjourned.

April 7th, 1909.—The meeting was called to order at 8.30 P. M.; President Onward Bates, in the chair; Chas. Warren Hunt, Secretary; and present, also, 111 members and 24 guests.

The minutes of the meetings of February 17th and March 3d, 1909, were approved as printed in *Proceedings* for March, 1909.

A paper by E. E. Howard, Assoc. M. Am. Soc. C. E., entitled "The Sixth Street Viaduct, Kansas City," was presented by the Secretary, and illustrated with lantern slides.

Written communications on the paper, from Messrs. Victor H. Cochrane and Daniel Bontecou, were read by the Secretary, and the subject was discussed orally by Messrs. G. H. Pegram and O. E. Mogensen.

The Secretary announced the election of the following candidates by the Board of Direction on April 6th, 1909:

As Members.

William Pope Anderson, Cincinnati, Ohio.
George Parker Bard, Boonton, N. J.
Charles Adrian Hammond, Mount Vernon, N. Y.
John Byers Holbrook, New York City.
Thomas John Morrison, Rochester, N. Y.
William Welcome Peabody, White Plains, N. Y.
George Perrine, New York City.
Frederick George Ray, Detroit, Mich.
Walter Britton Rittenhouse, Duluth, Minn.
Walter William Schlecht, Washington, D. C.
David Charles Serber, New York City.
Howard Eveleth Stevens, St. Paul, Minn.
Carlton Carpenter Witt, Sioux Falls, S. Dak.

As Associate Members.

Alfred Curtis Ackenheil, Aspinwall, Pa.
George Wishart Butz, Schuylkill Haven, Pa.
Frederick Charles Carstarphen, Denver, Colo.
James William Elliott, Burlington, Vt.
Charles Needham Forrest, Maurer, N. J.
Felder Furlow, Manãos, Brazil.
James Cowan Green, Albany, N. Y.
Jesse Warren Ijams, Albany, N. Y.
Maro Johnson, Chicago, Ill.
Samuel Reynolds Jones, New York City.
Arthur Stephen Lewis, Chicago, Ill.
Carl Bertram Lindholm, Russell, Mass.
Oscar Charles Merrill, Berkeley, Cal.
Fred Burgess Nelson, North Olmsted, Ohio.

George Bruce Palmer, Pittsburg, Pa.
David Thomas Pitkethly, Brooklyn, N. Y.
Frederic Adams Reimer, East Orange, N. J.
Thomas Riggs, Jr., Washington, D. C.
Donald Hubbard Sawyer, Seattle, Wash.
Thomas Arthur Smith, Jacksonville, Fla.
Elihu William Stevens, Hackensack, N. J.
Harold Lyell Stevens, Houston, Tex.
Robert Summers Stockton, Glendive, Mont.
Omner Jay Turley, Aztec, N. Mex.
Edmund Boyd Ulricu, Reading, Pa.
John Henry Leon Vogt, Louisville, Ky.
Adelbert Alonzo Weiland, Pueblo, Colo.
Lewis Maxwell Young, East Boston, Mass.

As Associate.

Frederick Eugene Townsend, New York City.

As Juniors.

Lowrey Wallace Anderson, Dallas, Tex.
Willard Townshend Chevalier, New York City.
William Franklin Collar, Princeton, Mich.
Herbert McMillen Dibert, Troy, N. Y.
James Calvin Foss, Jr., Wailuku, Hawaii.
Alvin Leroy Gilmore, New York City.
Chester Mason Gould, Cold Spring on Hudson, N. Y.
Edward Parmelee Hamilton, Havana, Cuba.
Benjamin Earl Jones, Washington, D. C.
Harold Vincent Joslin, Norfolk, Va.
Frank Edward Reed, Carmel, N. Y.
Patricio Andres Suarez y Cordoves, Santiago de Cuba, Cuba.
Charles Leopold Walker, Ithaca, N. Y.
James Madison Warner, Chicago, Ill.
George Lansing Wentworth, New York City.

The Secretary announced the transfer of the following candidates by the Board of Direction on April 6th, 1909:

From Associate Member to Member.

Frederick Auryansen, Jamaica, N. Y.
George Herbert Brazer, Boston, Mass.
Myron Samuel Falk, New York City.
Julian Hastings Granbery, New York City.
Joseph Jacobs, Portland, Ore.
Edwin Seton Jarrett, New York City.

Oscar Francis Lackey, Baltimore, Md.
Henry Bennett Machen, New York City.
John Wiley Miles, City of Mexico, Mex.
Henry Lewis Oestreich, New York City.
Howard Flanders Taylor, Kansas City, Mo.
Albert Lowry Webster, New York City.

From Junior to Associate Member.

Alexander Seymour Ackerman, Sandwich, Mass.
William Godfrey Arn, Corinth, Miss.
Collingwood Bruce Brown, Jr., Montreal, Canada.
Henry Avery Campbell, San Francisco, Cal.
Robert Edward Carrick, New York City.
Edward Christian Dicke, St. Louis, Mo.
Malcolm Elliott, Culebra, Canal Zone, Panama.
Thomas Lilly Fountain, Houston, Tex.
Thomas Hagen Gronwall, Pittsburg, Pa.
Alexander Sylvester Hamill, Jersey City, N. J.
William Heer, Jr., Tuscaloosa, Ala.
Leslie McHarg, New York City.
Charles Wesley Malcolm, Urbana, Ill.
Stanley Alfred Miller, Azua, Santo Domingo.
Frederick Horace Tibbetts, Berkeley, Cal.
John Hough Wickersham, Lancaster, Pa.

From Junior to Associate.

Henry Rogers Codwise, Brooklyn, N. Y.

The Secretary announced the following deaths:

James Eager Willard, elected Member May 1st, 1889; died March 8th, 1909.

Julius William Schaub, elected Junior November 5th, 1884; Member October 6th, 1886; died March 30th, 1909.

Adjourned.

OF THE BOARD OF DIRECTION

(Abstract)

March 2d, 1909.—President Bates in the chair; Chas. Warren Hunt, Secretary; and present, also, Messrs. Andrews, Brackett, Christie, Churchill, Hazen, Knap, Pegram, Schneider, Stearns, Swensson, Thompson, and Tillson.

Ballots for Membership were canvassed, and 11 Members, 13 Associate Members, 1 Associate, and 12 Juniors were elected, and 3 Juniors were transferred to the grade of Associate Member.

The Constitution adopted by the Colorado Association of Members of the American Society of Civil Engineers was approved.

The Secretary was authorized to issue a circular calling attention to a meeting for the consideration of the subject of the Conservation of Natural Resources to be held March 24th, 1909, at the Engineering Building.

Eight Associate Members were transferred to the grade of Member.

Applications were considered and other routine business transacted.

Adjourned.

April 6th, 1909.—President Bates in the chair; Chas. Warren Hunt, Secretary; and present, also, Messrs. Andrews, Brackett, Endicott, Kittredge, Noble, Pegram, Stuart, Swain, Swensson, Thompson, Tillson, and Williams.

Ballots for Membership were canvassed, and 13 Members, 28 Associate Members, 1 Associate, and 15 Juniors were elected, 16 Juniors were transferred to the grade of Associate Member, and 1 Junior was transferred to the grade of Associate.

The following resolutions were adopted:

Whereas, the accommodations available at the place of the coming Annual Convention are unlikely to exceed those needed for our own members, it is resolved that the Society do not invite any other National Society to participate in the said Convention.

Resolved, that members be informed that it is desirable that only those of their immediate family should be included in the attendance.

Resolved, that the Secretary be instructed, at the Annual Convention, not to enter on the list of those in attendance as guests any one who is not a member of the immediate family of a member of the Society.

The Secretary was authorized to issue to all members of the Society a pamphlet containing the addresses delivered at the joint meeting of Engineers, held March 24th, on the Conservation of Natural Resources.

Twelve Associate Members were transferred to the grade of Member.

Applications were considered and other routine business transacted.

Adjourned.

ANNOUNCEMENTS

The House of the Society is open from 9 A. M. to 10 P. M., every day, except Sundays, Fourth of July, Thanksgiving Day, and Christmas Day.

MEETINGS

Wednesday, May 5th, 1909.—8.30 P. M.—Two papers will be presented for discussion, as follows: "A System of Cost Keeping," by Myron S. Falk, Assoc. M. Am. Soc. C. E.; and "The Design of Elevated Tanks and Stand-Pipes," by C. W. Birch-Nord, Assoc. M. Am. Soc. C. E.

These papers were printed in *Proceedings* for March, 1909.

Wednesday, May 19th, 1909.—8.30 P. M.—At this meeting two papers will be presented for discussion as follows: "The Sewer System of San Francisco, and a Solution of the Storm-Water Flow Problem," by C. E. Grunsky, M. Am. Soc. C. E.; and "Some Extensive Railroad Surveys, and Their Cost per Mile," by W. S. McFetridge, M. Am. Soc. C. E.

Mr. Grunsky's paper was printed in *Proceedings* for March, 1909, and that by Mr. McFetridge appears in this number of *Proceedings*.

June 2d, 1909.—8.30 P. M.—Two papers will be presented for discussion as follows: "Tests of Built-Up Steel and Wrought-Iron Compression Pieces," by Arthur N. Talbot, M. Am. Soc. C. E., and Herbert F. Moore, Esq.; and "Caisson Disease and Its Prevention," by Henry Japp, M. Am. Soc. C. E.

These papers are printed in this number of *Proceedings*.

ANNUAL CONVENTION

The Forty-first Annual Convention of the Society will be held at the Mount Washington Hotel, Bretton Woods, N. H., from July 6th to July 9th, 1909, inclusive.

The general arrangements for the Convention are in the hands of the following Committees:

COMMITTEE OF THE BOARD OF DIRECTION

F. W. Hodgdon,

G. W. Tillson, Chas. Warren Hunt.

LOCAL COMMITTEE

J. W. Ellis,

A. W. Dean, J. F. Stevens,
H. W. Hayes, S. E. Tinkham,
George A. Kimball, H. D. Woods,

CONSERVATION OF NATURAL RESOURCES

A meeting of engineers was held in the Engineering Societies Building, 29 West 39th Street, New York City, at 8 P. M. on Wednesday, March 24th, 1909.

This meeting was called jointly by the four National Engineering Societies, each having selected one of its members to deliver an address on the Conservation of Natural Resources.

Dr. James Douglas, Past-President of the American Institute of Mining Engineers, presided. A telegram from President William H. Taft was read, and Dr. Douglas addressed the meeting.

Addresses were then delivered as follows:

"The Conservation of Water,"
John R. Freeman, M. Am. Soc. C. E.;
"The Conservation of Natural Resources by Legislation,"
R. W. Raymond, M. Am. Inst. M. E.;
"The Waste of Our Natural Resources by Fire,"
Charles Whiting Baker, M. Am. Soc. M. E., and
"Electricity and the Conservation of Energy,"
Lewis B. Stillwell, M. Am. Inst. E. E.

These addresses will not be published in *Proceedings,* but are being printed and will be mailed, in separate pamphlet form, to all members of the Society.

THE JOHN FRITZ MEDAL

The John Fritz Medal was established by the professional associates and friends of John Fritz, of Bethlehem, Pennsylvania, U. S. A., on August 21st, 1902, his eightieth birthday, to perpetuate the memory of his achievements in industrial progress. The rules governing the award provide that the medal shall be given "for notable scientific or industrial achievement, and that there shall be no restriction on account of nationality or sex." Awards may be made annually by a Board of sixteen, appointed or chosen in equal numbers from the membership of the four National Engineering Societies, the American Society of Civil Engineers, the American Institute of Mining Engineers, the American Society of Mechanical Engineers, the American Institute of Electrical Engineers.

The awards made to date have been as follows:

First award, January 20th, 1905, to Lord Kelvin for his work in cable telegraphy and other scientific attainments.

Second award, January 19th, 1906, to George Westinghouse for the invention and development of the air brake.

Third award, January 18th, 1907, to Alexander Graham Bell for the invention and introduction of the telephone.

Fourth award, January 17th, 1908, to Thomas Alva Edison for the invention of the duplex and quadruplex telegraph; the phonograph; the development of a commercially practical incandescent lamp; the development of a complete system of electric lighting, including dynamos, regulating devices, underground system protective devices and meters.

Fifth award, January 16th, 1909, to Charles T. Porter for his work in advancing the knowledge of steam engineering and in improvements in engine construction.

Board by which the last award was made:

E. G. Spilsbury, Chairman; Chas. Warren Hunt, Secretary; C. C. Schneider, Frederic P. Stearns, Geo. H. Benzenberg, James Douglas, Charles Kirchhoff, E. E. Olcott, John E. Sweet, Henry R. Towne, Ambrose Swasey, F. R. Hutton, John W. Lieb, Jr., S. S. Wheeler, Samuel Sheldon, H. G. Stott.

The medal was presented to Charles Talbot Porter on Tuesday evening, April 13th, 1909, in the Engineering Societies Building, New York City.

The presentation was made by E. Gybbon Spilsbury, Past-President, Am. Inst. E. E.; M. Am. Soc. C. E.; Chairman of the Board of Award; and the following addresses were delivered:

"The Debt of Modern Industrial Civilization to the Steam Engine as a Source of Power."

W. F. M. Goss, M. Am. Soc. M. E.; Assoc. Am. Inst. E. E.; Dean of the College of Engineering of the University of Illinois.

"The Debt of the Modern Steam Engine to Charles T. Porter."

F. R. Hutton, Hon. Sec., Am. Soc. M. E.

"The Debt of the Era of Steel to the High-Speed Engine."

Robert W. Hunt, M. Am. Soc. C. E.; Past-President, Am. Soc. M. E.; Past-President, Am. Inst. M. E.

"The Debt of the Era of Electricity to the High-Speed Steam Engine."

Frank J. Sprague, Past-President, Am. Inst. E. E.; M. Am. Soc. C. E.

PAPERS AND DISCUSSIONS

The first volume of *Transactions* for 1909 (Vol. LXII) has been issued. There will be three additional volumes issued during the year.

It is hoped that members and others who take part in the discussion of the papers presented will revise their remarks promptly, and that all written communications from those who cannot attend the meetings will be sent in at the earliest possible date after the issue of a paper in *Proceedings*. The issue of volumes of *Transactions* is

dependent on the closing of discussions, and the co-operation of the membership will now be more necessary in this matter than heretofore, because four volumes are to be issued during the year instead of two, and, to accomplish this, a definite date of issue for each has been established. It is expected that the second volume of 1909 will be issued on June 30th, and the third and fourth on September 30th and December 31st, respectively.

All papers accepted by the Publication Committee are classified by the Committee with respect to their availability for discussion at meetings.

Papers, which, from their general nature, appear to be of a character suitable for oral discussion, will be published as heretofore in *Proceedings,* and set down for presentation to a future meeting of the Society, and, on these, oral discussion, as well as written communications, will be solicited.

All papers which do not come under this heading, that is to say, those which, from their mathematical or technical nature, in the opinion of the Committee, are not adapted to oral discussion, will not be scheduled for presentation to any meeting. Such papers will be published in *Proceedings* in the same manner as those which are to be presented at meetings, but written discussions, only, will be requested for subsequent publication in *Proceedings* and with the paper in the volumes of *Transactions.*

PRIVILEGES OF ENGINEERING SOCIETIES EXTENDED TO MEMBERS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

Members of the American Society of Civil Engineers will be welcomed by the following Engineering Societies, both to the use of their Reading Rooms and at all Meetings:

American Institute of Mining Engineers, 29 West Thirty-ninth Street, New York City.

Associação dos Engenheiros Civis Portuguezes, Lisbon, Portugal.

Australasian Institute of Mining Engineers, Melbourne, Victoria, Australia.

Boston Society of Civil Engineers, 715 Tremont Temple, Boston, Mass.

Brooklyn Engineers' Club, 197 Montague Street, Brooklyn, N. Y.

Canadian Society of Civil Engineers, 877 Dorchester Street, Montreal, Que., Canada.

Civil Engineers' Society of St. Paul, St. Paul, Minn.

Cleveland Engineering Society, 718 Caxton Building, Cleveland, Ohio.

Cleveland Institute of Engineers, Middlesbrough, England.

Colorado Association of Members, Am. Soc. C. E., 235 Equitable Building, Denver, Colo.

Engineers' and Architects' Club of Louisville, Ky., 303 Norton Building, Fourth and Jefferson Streets, Louisville, Ky.

Engineers' Club of Baltimore, Baltimore, Md.

Engineers' Club of Central Pennsylvania, Corner Second and Walnut Streets, Harrisburg, Pa.

Engineers' Club of Minneapolis, 17 South Sixth Street, Minneapolis, Minn.

Engineers' Club of Philadelphia, 1317 Spruce Street, Philadelphia, Pa.

Engineers' Club of St. Louis, 3817 Olive Street, St. Louis, Mo.

Engineers' Club of Toronto, 96 King Street, West, Toronto, Ont., Canada.

Engineers' Society of Western Pennsylvania, 803 Fulton Building, Pittsburg, Pa.

Institute of Marine Engineers, 58 Romford Road, Stratford, London, E., England.

Institution of Engineers of the River Plate, Buenos Aires, Argentine Republic.

Institution of Naval Architects, 5 Adelphi Terrace, London, W. C., England.

Junior Institution of Engineers, 39 Victoria Street, Westminster, S. W., London, England.

Koninklijk Instituut van Ingenieurs, The Hague, The Netherlands.

Louisiana Engineering Society, 604 Tulane-Newcomb Building, New Orleans, La.

Memphis Engineering Society, Memphis, Tenn.

Midland Institute of Mining, Civil and Mechanical Engineers, Sheffield, England.

Montana Society of Engineers, Butte, Montana.

North of England Institute of Mining and Mechanical Engineers, Newcastle-upon-Tyne, England.

Oesterreichischer Ingenieur- und Architekten-Verein, Eschenbachgasse 9, Vienna, Austria.

Pacific Northwest Society of Engineers, 617-618 Pioneer Building, Seattle, Wash.

Rochester Engineering Society, Rochester, N. Y.

Sachsischer Ingenieur- und Architekten-Verein, Dresden, Germany.

Sociedad Colombiana de Ingenieros, Bogota, Colombia.

Societe des Ingenieurs Civils de France, 19 Rue Blanche, Paris, France.

Society of Engineers, 17 Victoria Street, Westminster, S. W., England.

Svenska Teknologföreningen, Brunkebergstorg 18, Stockholm, Sweden.

Tekniske Forening, Vestre Boulevard 18–1, Copenhagen, Denmark.

Western Society of Engineers, 1737 Monadnock Block, Chicago, Ill.

SEARCHES IN THE LIBRARY

In January, 1902, the Secretary was authorized to make searches in the Library, upon request, and to charge therefor the actual cost to the Society for the extra work required. Since that time many searches have been made, and bibliographies and other information on special subjects furnished.

The resulting satisfaction, to the members, who have made use of the resources of the Society in this manner, has been expressed frequently, and leaves little doubt that, if it were generally known to the membership that such work would be undertaken, many would avail themselves of it.

The cost is trifling compared with the value of the time of an engineer who looks up such matters himself, and the work can be performed quite as well, and much more quickly, by persons familiar with the Library.

In asking that such work be undertaken, members should specify clearly the subject to be covered, and whether references to general books only are desired, or whether a complete bibliography, involving search through periodical literature, is desired.

In reference to this work the Appendix* to the Annual Report of the Board of Direction for the year ending December 31st, 1906, contains a summary of all searches made to that date.

* *Proceedings*, Vol. XXXIII, p. 20 (January, 1907).

ACCESSIONS TO THE LIBRARY

(From March 10th to April 12th, 1909.)

DONATIONS.*

SURVEYOR'S HAND BOOK.

By T. U. Taylor, M. Am. Soc. C. E. Roan, 6¾ x 4½ in., illus., 13 + 310 pp. Chicago and New York, The Myron C. Clark Publishing Co., 1908. $2.00 net.

The purpose of the author, as stated in his preface, has been to prepare, for the use of the surveyor in the field, a book of convenient size and scope, and one containing all the essentials for ordinary surveying. The traverse table has been omitted, as, in the author's opinion, small traverse tables are of no avail in an exact survey and the large ones are books in themselves. Several of the chapters have short bibliographies at the end. The Contents are: Chain Surveying; Compass Surveying; Transit Surveying; Calculation of Areas; Division of Land; Leveling; Topographic Survey; Railroad Surveying; Earthwork; City Surveying; Plotting and Lettering; Government Surveying; Trigonometric Formulas; Tables; Index.

ENGINEERING WORK IN TOWNS AND CITIES.

By Ernest McCullough. Second Edition. Cloth, 8 x 5½ in., illus., 8 + 502 pp. Chicago and New York, The Myron C. Clark Publishing Co., 1908. $3.00 net.

This work is written for two classes of town officials, those with no technical education, as councilmen, etc., and those having more or less technical training, as town or city engineers. In the first edition each subject was treated in two chapters, one from each standpoint, but in this edition the two treatments have been combined so that every subject is complete for all classes of readers in the chapter devoted to it. Where errors were found, the preface states, they have been corrected in this edition, some new matter has been added, and the chapter on Reinforced Concrete entirely rewritten. The author states also that much of the information is the accumulation of many years' experience, and that he believes some of it to be here printed for the first time, as it has been picked up in the field, over the drawing board, at engineers' conventions and in conversation with experienced and practical men. The Contents are: The City Engineer and His Duties; Roads and Streets; Walks, Curbs and Gutters; Street Pavements; Sanitation; Drainage and Sewerage; Water Supply; Concrete; Contracts and Specifications; Miscellaneous Data; Office Systems; City Engineer's Records; Field Work; Engineering Data; Index.

THE DESIGN AND EQUIPMENT OF SMALL CHEMICAL LABORATORIES.

By Richard K. Meade. Cloth, 9¼ x 6 in., illus., 136 pp. Chicago, The Chemical Engineer Publishing Co., 1908. $2.00 net.

The author states that while this book is intended primarily as suggestions to young and inexperienced chemists, he hopes experienced chemists will find in it some points of use and help. The firms making the various forms of apparatus described are mentioned by name for the convenience of the reader. The Contents are: General Features; Hoods; Sinks and Water Supply; Desks; Table and Apparatus for Rapid Filtrations, etc.; Ignition Table and Apparatus for Ignition; Table and Apparatus for Titrations; Balance Support, Balance, and Accessories; Heating Appliances; Preparation of Distilled Water; Apparatus for Electro-chemical Analysis; Sampling Appliances; Assay Furnaces and Accessories; Miscellaneous Laboratory Equipment; Index.

THE ENGINEERING INDEX ANNUAL FOR 1908.

Compiled from the Engineering Index Published Monthly in *The Engineering Magazine* during 1908. Cloth, 9½ x 6½ in., 14 + 437 pp. New York and London, The Engineering Magazine, 1909. $2.00.

This is the seventh volume of the Engineering Index issued since the work was first undertaken by the late J. B. Johnson, M. Am. Soc. C. E., and the third since

*Unless otherwise specified, books in this list have been donated by the publisher.

the index became an annual. Taken together these volumes form a continuous index to engineering and technical literature from the beginning of 1884. The arrangement of the index is still the same as that published monthly in *The Engineering Magazine*. The articles are first grouped under the great divisions of engineering practice—Civil, Mechanical, Electrical, Mining, etc.—and under these again sub-grouped according to the recognized special divisions of each field, the final arrangement under these being strictly alphabetical. The 1908 Annual includes about two hundred and fifty publications, about three-fourths of which are printed in English, the others being in German, French, Spanish, Italian, and Dutch. Following each entry is a brief descriptive note defining the scope and purport of the article. Serials are indexed under the first installment except in the case of short articles, which are indexed entire. The Annual is carried through 1908, but a continuation of it is issued monthly in *The Engineering Magazine*.

THEORY OF SOLID AND BRACED ELASTIC ARCHES.

By William Cain, M. Am. Soc. C. E. Second Edition, Revised and Enlarged. Boards, 6 x 3¾ in., illus., 8 + 100 pp. New York, D. Van Nostrand Company, 1909. 50 cents. (Donated by the Author.)

This work has been, the author states, almost entirely rewritten, and treats in great detail of elastic arches of variable cross-section, particularly concrete and reinforced concrete arches, the aim being, not only to present the theory in a simple, convincing manner, but to furnish the computer with several examples worked out in full, so that he shall find no difficulty in applying the theory rapidly and with certainty, to similar arches. The Chapter headings are: Equilibrium Polygons, Elastic Theory; Concrete Arch, Both Graphical and Analytical Solutions; Three-Centered Arch, Algebraic Solution in Full; Temperature and Allied Stresses; Reinforced Concrete Arch, Method of Single Loads; Arches with Two or Three Hinges, Braced Arches.

UNIVERSITY OF LONDON, CHADWICK LECTURES, SESSION 1907-08.

Containing Lectures on the Engineering Aspect of Recent Advances in Connection with Sewering. By W. D. Scott-Moncrieff. Paper, 8½ x 5½ in., 79 pp. London, St. Bride's Press, Ltd., 1909. Two shillings net.

This series of four lectures was delivered at the University of London under the endowment derived from the Chadwick Trust, which requires that the lectures shall have special reference to recent advances in hygiene and municipal engineering. Lecture I is historical and introductory, and takes up the early unsanitary condition of England, and summarizes the progress made, reports of committees, etc.; Lecture II deals with the functions of the physician, chemist, bacteriologist, and engineer in sanitary progress, and the special work of the engineer in connection with the sewering of houses and streets; Lecture III is on sewage disposal; Lecture IV discusses what the engineer, engaged in sewage disposal work, should know with regard to chemistry, bacteriology, etc.

THE RAILROAD SIGNAL DICTIONARY

An Illustrated Vocabulary of Terms Which Designate American Railroad Signals, Their Parts, Attachments, and Details of Construction, With Descriptions of Methods of Operation and Some Illustrations of British Signals and Practice. Comp. for the Railway Signal Association by Braman B. Adams and Rodney Hitt, Jun. Am. Soc. C. E. Morocco, 12 x 8½ in., illus., 10 + 36 + 472 pp. New York and Chicago, Railroad Age-Gazette; London, Railway Gazette, 1908. $6.00.

This work consists of thirty-two pages of dictionary text and more than four hundred pages of illustrations interspersed with a small amount of text. At the end of each dictionary article there are references to the figures in the album part of the volume. The illustrations are grouped in five parts: Signal Indications; Block Signals; Highway Crossing Signals; Interlocking; Block and Interlocking Accessories. There are classified and alphabetical indexes given at the beginning of the album portion of the volume, making the collection of engravings accessible to the user.

Gifts have also been received from the following:

Am. Gas Inst. 1 bound vol.
Argentine Republic-Ministerio de Obras Publicas. 1 pam.
Arizona-Agri. Exper. Station. 5 pam.
Arlington, Mass.-Town Clerk. 6 vol.
Arnold, Bion J. 3 pam.
Australia-Commonwealth Statistician. 3 pam.
Belzner, Theodore. 2 pam.
Berlin Kgl. Technische Hochschule. 1 pam.
Boston, Mass.-Health Dept. 1 bound vol., 1 pam.
Brookline, Mass.-Water Dept. 1 pam.
Bureau of Explosives. 3 pam.
California-State Forester. 2 pam.
California-State Harbor Commrs. 3 pam.
Canada-Dept. of Rys. and Canals. 1 vol.
Canadian Min. Inst. 2 pam.
Canadian Soc. of Civ. Engrs. 2 pam.
Chicago, Ill.-Dept. of Smoke Prevention. 3 pam.
Chicago, Indiana & Southern R. R. Co. 1 pam.
Civ. Engrs.' Soc. of St. Paul, Minn. 1 pam.
Cleveland Eng. Soc. 1 pam.
Colorado-State Engr. 2 bound vol.
Columbus, Ohio-Chf. Engr. of Dept. of Public Service. 1 pam.
Detroit, Mich.-Board of Water Commrs. 2 pam.
Eastern Bengal and Assam-Public Works Dept. 1 pam.
Fitchburg, Mass.-City Engr. 1 pam.
Glasgow & South-Western Ry. Co. 1 pam.
Graduates Club of New York City. 1 bound vol.
Great Central Ry. Co. 1 pam.
Great North of Scotland Ry. Co. 2 pam.
Green Bay & Western R. R. Co. 1 pam.
Greene, G. S. 19 bound vol., 24 pam.
Highland Ry. Co. 1 pam.
John Crerar Library. 1 pam.
Johnstown, N. Y.-Board of Water Commrs. 1 pam.
Lake Erie & Western R. R. Co. 1 pam.
Lake Shore & Mich. Southern Ry. Co. 1 pam.
Leominster, Mass.-Water Board. 1 pam.
Maine-Commr. of Highways. 1 bound vol.
Merchants' Assoc. of New York. 2 pam.
Mich. Central R. R. Co. 1 pam.
Michigan Technic. 1 pam.
National Assoc. of Cotton Mfrs. 1 bound vol.
Nebraska-State Engr. 7 bound vol.
New Jersey-Commr. of Public Roads. 1 bound vol.
New Jersey-State Board of Health. 1 pam.
New Jersey San. Assoc. 1 pam.
New York City-Board of Estimate and Apportionment. 1 pam.
New York City-Mayor. 1 vol.
New York State-Public Service Comm., First Dist. 1 pam.
New York State-Public Service Comm., Second Dist. 2 bound vol., 1 pam., 1 map.
New York-State Engr. and Surv. 1 map.
New York Central & Hudson R. R. Co. 1 pam.
New York City Record. 1 bound vol.
Newark, N. J.-Board of Street and Water Commrs. 2 pam.
Newhall, Henry. 37 pam.
North, Edward P. 2 bound vol.
North British Ry. Co. 1 pam.
Nova Scotia Min. Soc. 1 vol.
Oberlin College Library. 1 pam.
Ohio-State Board of Health. 1 bound vol.
Ohio State Univ. 1 vol., 1 pam.
Oesterreichischer Ingenieur- und Architekten-Verein. 4 pam.
Pennsylvania R. R. Co. 1 pam.
Philippine Islands-Bureau of Forestry. 3 pam.
Philippine Islands-Bureau of Science. 2 pam.
Punjab, India-Irrig. Dept. 1 bound vol.
Randle, George N. 6 pam.
Rensselaer Polytechnic Inst. 1 pam.
Rhode Island-R. R. Commr. 1 bound vol.
Royal Philosophical Soc. of Glasgow. 1 vol.
St. Louis, Mo., Merchants' Exchange. 1 pam.
St. Paul, Minn.-Board of Water Commrs. 1 pam.
St. Paul, Minn.-City Comptroller. 1 bound vol.
Saskatchewan, Canada-Dept. of Public Works. 1 vol.
Snare & Triest Co. 1 bound vol.
Société Belge des Ingénieurs et des Industriels. 4 pam.
Soc. of Engrs. 1 pam.
Soc. of Naval Archts. and Marine Engrs. 1 pam.
Texas-Tax Commr. 1 pam.
Texas & Pacific Ry. Co. 1 pam.
Transvaal-Mines Dept. 1 vol.
U. S.-Bureau of Insular Affairs. 1 vol.
U. S.-Bureau of Labor. 1 bound vol.
U. S.-Bureau of Standards. 4 pam.
U. S.-Chief of Engrs. 2 vol.
U. S.-Coast and Geodetic Survey. 1 vol., 1 pam.
U. S.-Copyright Office. 1 pam.
U. S.-Dept. of Agriculture. 1 pam.
U. S.-Geol. Survey. 2 bound vol., 4 vol., 5 pam.
U. S.-Geol. Survey-Technologic Branch. 1 pam.
U. S.-Interstate Commerce Comm. 3 bound vol.
U. S.-Isthmian Canal Comm. 1 pam.
U. S.-Office of Library and Naval War Records. 1 pam.
U. S. Steel Corporation. 1 pam.
Univ. of Cal.-Univ. Press. 9 vol.
Univ. of Illinois-Eng. Exper. Station. 1 bound vol.
Univ. of Vermont and State Agri. Coll. 1 pam.
Verein Deutscher Eisenhüttenleute. 1 vol.
Waltham, Mass.-Water Dept. 1 pam.
Wauters, Carlos. 4 pam.
Wellington, New Zealand-Harbour Board. 1 pam.
Western Australia-Geol. Survey. 2 pam.
Wilgus, William J. 1 bound vol.
Wis. Valley Improvement Co. 1 pam.

BY PURCHASE

Electrical Illuminating Engineering. By W. E. Barrows, Jr. McGraw Publishing Co., New York, 1908.

Design and Construction of Ships. By John Harvard Biles. Vol. 1, Calculations and Strength. Charles Griffin & Co., Ltd., London, 1908.

The Railway Locomotive. What It Is and Why It Is What It Is. By Vaughan Pendred. Archibald Constable & Co., Ltd., London, 1908.

Atlas of Canada. Prepared under the Direction of James White, Geographer. Department of the Interior, Canada, 1906.

Public Health. Papers and Reports Presented at the Thirty-fifth Annual Meeting of the American Public Health Association, Atlantic City, New Jersey, Sept. 30th, Oct. 1st, 2d, 3d, 4th, 1907. Vol. XXXIII, Parts I and II. Columbus, Ohio, Fred. J. Heer, 1908.

Handbuch über Triebwagen für Eisenbahnen. Von C. Guillery. R. Oldenbourg, München und Berlin, 1908.

Über den Querschnitt der Staumauern. Von Ferd. Platzmann. Fortschritte der Ingenieurwissenschaften. Wilhelm Engelmann, Leipzig, 1908.

Guide to Sanitary Inspections. By W. P. Gerhard. Ed. 4, rev. and enl. John Wiley & Sons, New York; Chapman & Hall, London, 1909.

High-Tension Underground Electric Cables: A Practical Treatise for Engineers. By Henry Floy. Electrical Publishing Co., New York, 1909.

SUMMARY OF ACCESSIONS

(From March 9th to April 12th, 1909)

Donations (including 56 duplicates)	272
By purchase	10
Total	282

MEMBERSHIP

ADDITIONS

(March 10th to April 13th, 1909)

MEMBERS		Date of Membership
ALLARD, THOMAS THROP. 101 Obispo St., Havana, Cuba....		Mar. 2, 1909
BASCOM, HARRY FRANKLIN. Engr. (Bascom & Sieger), 603 Allentown Bank Bldg., Allentown, Pa.....	Jun.	Dec. 5, 1899
	Assoc. M.	Sept. 4, 1901
	M.	Mar. 2, 1909
BRAZER, GEORGE HERBERT. (J. R. Worcester & Co., Cons. Engrs.), 79 Milk St., Boston, Mass.....	Assoc. M.	Nov. 1, 1905
	M.	April 6, 1909
CRELLIN, EDWARD WEBSTER. Pres., Des Moines Bridge & Iron Works, Des Moines, Iowa.....		Mar. 2, 1909
FALK, MYRON SAMUEL. Cons. Engr., 30 Church St. (Res., 44 West 53d St.), New York City.....	Jun.	Feb. 4, 1902
	Assoc. M.	Oct. 4, 1905
	M.	April 6, 1909
HOUSTON, GAVIN NELSON.. Deputy State Engr., State House, Denver, Colo.....	Jun.	May 4, 1897
	Assoc. M.	Nov. 7, 1900
	M.	Feb. 2, 1909
KELSEY, LOUIS CURTIS. City Engr., Salt Lake City, Utah..		Mar. 2, 1909
LACKEY, OSCAR FRANCIS. Harbor Engr., Baltimore, Md.....	Assoc. M.	May 4, 1904
	M.	April 6, 1909
MACHEN, HENRY BENNETT. Asst. Engr., Dept. of Water Supply (Res., 315 West 97th St.), New York City.....	Jun.	May 2, 1899
	Assoc. M.	Feb. 1, 1905
	M.	April 6, 1909
MCMURTRY, GUY. Yuba City, Cal.....		Mar. 2, 1909
MORRISON, THOMAS JOHN. 112 Cutler Bldg., Rochester, N. Y.		April 6, 1909
NELSON, JAMES AUGUSTUS. 800 Westinghouse Bldg., Pittsburg, Pa.....		Mar. 2, 1909
OESTREICH, HENRY LEWIS. Asst. Engr., Public Service Comm., 154 Nassau St. (Res., 2092 Bathgate Ave.), New York City....	Jun.	Oct. 4, 1892
	Assoc. M.	Dec. 6, 1899
	M.	April 6, 1909
PALMER, JOHN GEARY. Barge Canal Office, Montezuma, N. Y.....		Oct. 7, 1908
RAY, FREDERICK GEORGE. 33 Campau Bldg., Detroit, Mich..		April 6, 1909
SERBER, DAVID CHARLES. Structural Engr., Henry Steers, Inc., 17 Battery Pl., New York City.....		April 6, 1909
SMITH, GILMAN WALTER. Div. Erecting Mgr., Am. Bridge Co. of N. Y., Commercial National Bank Bldg., Chicago, Ill.....		Mar. 2, 1909
STEVENS, HOWARD EVELETH. Bridge Engr., N. P. Ry. Co., St. Paul, Minn.....		April 6, 1909
WEBSTER, ALBERT LOWRY. Cons. Engr., 82 Wall St., New York City.....	Jun.	Sept. 6, 1882
	Assoc. M.	June 3, 1891
	M.	April 6, 1909

MEMBERS (*Continued*)

		Date of Membership
WHITE, WILLIAM MACINTIRE. Engr., United States Steel Products Export Co., 30 Church St., Room 1320, New York City..	Assoc. M.	Jan. 2, 1901
	M.	Mar. 2, 1909
WITT, CARLTON CARPENTER. Engr., South Dakota R. R. Comm., Sioux Falls, S. Dak.....		April 6, 1909

ASSOCIATE MEMBERS

ACKERMAN, ALEXANDER SEYMOUR. Asst. Engr., Cape Cod Canal Co., Sandwich, Mass....	Jun.	April 30, 1907
	Assoc. M.	April 6, 1909
BLAIR, MCCREA PARKER. City Engr., City Hall, St. Boniface, Man., Canada.....		Feb. 2, 1909
BLAYLOCK, JOHN CHARLES. 1487 Wellington St., Chicago, Ill.		Mar. 2, 1909
BRENEMAN, PAUL BRUCE. Box 294, State College, Pa.....		Mar. 2, 1909
BUTZ, GEORGE WISHART. Civ. and Cons. Engr., Schuylkill Haven, Pa.....		April 6, 1909
CARRICK, ROBERT EDWARD. Engr., The Gen. Fireproofing Co., 257 East 133d St., New York City.....	Jun.	April 30, 1907
	Assoc. M.	April 6, 1909
DICKE, EDWARD CHRISTIAN. 1909 Hebert St., St. Louis, Mo.....	Jun.	May 3, 1904
	Assoc. M.	April 6, 1909
FEDERLEIN, WALTER GOTTLIEB. 2644 Marion Ave., New York City.....	Jun.	Jan. 2, 1906
	Assoc. M.	Feb. 2, 1909
FORREST, CHARLES NEEDHAM. Maurer, N. J.....		April 6, 1909
HAMILL, ALEXANDER SYLVESTER. 426 Montgomery St., Jersey City, N. J.....	Jun.	Dec. 28, 1900
	Assoc. M.	April 6, 1909
IJAMS, JESSE WARREN. Asst. Engr., N. Y. C. & H. R. R. R. Co., Room 32, Union Station, Albany, N. Y.....		April 6, 1909
JUDD, FRANK RHYMAL. 6543 Minerva Ave., Chicago, Ill....		Mar. 2, 1909
LEWIS, ARTHUR STEPHEN. 1341 Sheridan Rd., Chicago, Ill.		April 6, 1909
LINDHOLM, CARL BERTRAM. Room 372, South Station, Boston, Mass.....		April 6, 1909
MACARTNEY, MORTON. P. O. Box 2241, Spokane, Wash.....		Mar. 2, 1909
MCHARG, LESLIE. Contr., 165 Broadway, New York City.....	Jun.	Mar. 5, 1901
	Assoc. M.	April 6, 1909
MILLER, HIRAM. Care, Bolivia Ry. Co., La Paz (via Panama), Bolivia.....		Jan. 5, 1909
MOODY, JOSEPH ELBERT. 316 Burke Bldg., Seattle, Wash...		Oct. 7, 1908
MURPHY, JOHN JOSEPH. 70 Park Hill Ave., Yonkers, N. Y..		Mar. 2, 1909
NELSON, FRED BURGESS. North Olmsted, Ohio.....		April 6, 1909
PALMER, GEORGE BRUCE. Chf. Inspector, Penn. Lines W. of Pitts., 1106 Union Station, Pittsburg, Pa.....		April 6, 1909
PECK, MYRON HALL. Prof. of Civ. Eng., Imperial Pei Yang Univ., Hsi Ku, Tientsin, China.....		Jan. 5, 1909
POWELL, THOMAS JETT. 1227 E St., N. E., Washington, D. C.....		Mar. 2, 1909

ASSOCIATE MEMBERS (*Continued*)

		Date of Membership
RIGGS, THOMAS, JR. Coast and Geodetic Survey, 206 Federal Bldg., Seattle, Wash		April 6, 1909
RIGHTS, EUGENE JESSE. 3806 Ridge Ave., Wissahickon, Philadelphia, Pa		Mar. 2, 1909
SMITH, THOMAS ARTHUR. P. O. Box 807, Jacksonville, Fla		April 6, 1909
ULRICH, EDMUND BOYD. 303 Baer Bldg., Reading, Pa		April 6, 1909
WICKERSHAM, JOHN HOUGH. Lancaster, Pa	Jun.	May 6, 1902
	Assoc. M.	April 6, 1909
YOUNG, LEWIS MAXWELL. 119 Trenton St., East Boston, Mass		April 6, 1909

ASSOCIATE

CODWISE, HENRY ROGERS. Instr. in Eng., Poly. Inst. of Brooklyn, Brooklyn, N. Y., and Instr. in Eng., Field Work and Math., Cooper Union Schools of Science and Art, New York City	Jun.	April	2, 1901
	Assoc.	April	6, 1909

JUNIORS

BARNES, HARRY EVERETT. Box 113, Stone Ridge, Ulster Co., N. Y	Mar.	2, 1909
CHEVALIER, WILLARD TOWNSHEND. Care, C. T. & E. S. Co., 66 Lafayette St., New York City	April	6, 1909
DIBERT, HERBERT MCMILLEN. 10 First St., Troy, N. Y	April	6, 1909
EBERSPACHER, FRED. U. S. Engr. Office, Vicksburg, Miss	Mar.	2, 1909
PRUETT, GROVER CLEVELAND. Care, C., M. & St. P. Ry. Co., Miles City, Mont	Mar.	2, 1909
RAMSDELL, ROBERT LEROY. 1215 Caroline St., Houston, Tex	Oct.	6, 1908
ROSSELL, PAUL FRANCIS. 1201 N. Clayton St., Wilmington, Del	Mar.	2, 1909
SPOONER, CHARLES WILLETT. 205 New Engineering Bldg., Ann Arbor, Mich	Mar.	2, 1909
WENTWORTH, GEORGE LANSING. Gerard Hotel, 123 West 44th St., New York City	April	6, 1909
WITTSTEIN, HERMAN LEWIS. Care, A. L. Webster, 82 Wall St., New York City	Oct.	6, 1908
WYGANT, ROBERT CECIL. Newport, Ore	Oct.	6, 1908

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ASSOCIATE

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BELZNER, THEODORE. Insp. of Steel, Williamsburgh Bridge, Dept. of Bridges, 84 Broadway, Brooklyn, N. Y. (Res., 614 West 135th St., New York City).
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CORTRIGHT, EDWIN KEEN. Beverly, N. J.
COTTON, MILES PENNER. Engr. and Gen. Contr., 27 Davis Chambers, Vancouver, B. C., Canada.
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EAMES, HORACE LOVELL. Acting Dist. Engr., 1st Dist., Bureau of Public Works, Tuguegarao, Cagayan, Philippine Islands.
EWING, WILLIARD REA. 1142 S. Ridgway Ave., Chicago, Ill.
GOTWALS, JOHN CARL. Stillwater, N. Y.
HAMLIN, RALPH. Chf. Draftsman, Cummings Structural Concrete Co., McKees Rocks, Pa.
HILDER, FRAZER CROSWELL. Office of Indian Affairs, Dept. of the Interior, Washington, D. C.
HOPSON, EDWARD HAROLD. Asst. Engr., Board of Water Supply, New Paltz, N. Y.
JOHNSTON, ANDREW CRAWFORD. 73 Herriman Ave., Jamaica, N. Y.
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NIKOLITCH, MILAN. 224 Twenty-first Ave., Rich. Dis., San Francisco, Cal.
PIERCE, PAUL LEON. Culebra, Canal Zone, Panama.
RACKLE, OSCAR WILLIAM. 94 Angell St., Providence, R. I.
RICH, WILDER MELOY. U. S. Engr. Office, Sault Ste. Marie, Mich.
ROWE, WILFRED LINCOLN. 714 Van Buren St., Milwaukee, Wis.
SCHULTZ, CHARLES. Oakwood, Mo.
SMITH, ELROY GEORGE. 159 West 131st St., New York City.
THOMSON, FRED MORTON. Care, R. A. Thompson, Chf. Engr., "Wichita Falls Route," Wichita Falls, Tex.
WALKER, GEORGE JOHNSON. Draftsman with Heyl & Patterson, Contr. Engrs. (Res., 731 S. Negley Ave.), Pittsburg, Pa.
WATSON, GEORGE LINTON. Engr., United Pav. Co., Atlantic City, N. J.
WESTOVER, HENRY CHRISTOPHER. 1401 Felix St., St. Joseph, Mo.
WILCOX, FRANK LESLIE. Park Ave. Hotel, New York City.

FELLOW

WATSON, JAMES. Hopatcong House Landing, N. J.

DEATHS

BREEN, HOWARD. Elected Member, April 4th, 1888; died February 14th, 1909.

DEANS, CHARLES HERBERT. Elected Junior, December 3d, 1890; Associate Member, May 6th, 1896; died March 7th, 1909.

SCHAUB, JULIUS WILLIAM. Elected Junior, November 5th, 1884; Member, October 6th, 1886; died March 30th, 1909.

WILLARD, JAMES EAGER. Elected Member, May 1st, 1889; died March 8th, 1909.

MONTHLY LIST OF RECENT ENGINEERING ARTICLES OF INTEREST

(March 9th to April 12th, 1909)

NOTE.—*This list is published for the purpose of placing before the members of the Society, the titles of current engineering articles which can be referred to in any available engineering library, or can be procured by addressing the publication directly, the address and price being given wherever possible.*

LIST OF PUBLICATIONS

In the subjoined list of articles, references are given by the number prefixed to each journal in this list.

(1) *Journal*, Assoc. Eng. Soc., 31 Milk St., Boston, Mass., 30c.
(2) *Proceedings*, Engrs. Club of Phila., 1317 Spruce St., Philadelphia, Pa.
(3) *Journal*, Franklin Inst., Philadelphia, Pa., 50c.
(4) *Journal*, Western Soc. of Engrs., Monadnock Blk., Chicago, Ill.
(5) *Transactions*, Can. Soc. C. E., Montreal, Que., Canada.
(6) *School of Mines Quarterly*, Columbia Univ., New York City, 50c.
(7) *Technology Quarterly*, Mass. Inst. Tech., Boston, Mass., 75c.
(8) *Stevens Institute Indicator*, Stevens Inst., Hoboken, N. J., 50c.
(9) *Engineering Magazine*, New York City, 25c.
(10) *Cassier's Magazine*, New York City, 25c.
(11) *Engineering* (London), W. H. Wiley, New York City, 25c.
(12) *The Engineer* (London), International News Co., New York City, 35c
(13) *Engineering News*, New York City, 15c.
(14) *The Engineering Record*, New York City, 12c.
(15) *Railroad Age Gazette*, New York City, 15c.
(16) *Engineering and Mining Journal*, New York City, 15c.
(17) *Electric Railway Journal*, New York City, 10c.
(18) *Railway and Engineering Review*, Chicago, Ill., 10c.
(19) *Scientific American Supplement*, New York City, 10c.
(20) *Iron Age*, New York City, 10c.
(21) *Railway Engineer*, London, England, 25c.
(22) *Iron and Coal Trades Review*, London, England, 25c.
(23) *Bulletin*, American Iron and Steel Assoc., Philadelphia, Pa.
(24) *American Gas Light Journal*, New York City, 10c.
(25) *American Engineer*, New York City, 20c.
(26) *Electrical Review*, London, England.
(27) *Electrical World*, New York City, 10c.
(28) *Journal*, New England Water-Works Assoc., Boston, Mass., $1.
(29) *Journal*, Royal Society of Arts, London, England, 15c.
(30) *Annales des Travaux Publics de Belgique*, Brussels, Belgium.
(31) *Annales* de l'Assoc. des Ing. Sortis des Ecoles Spéciales de Gand, Brussels, Belgium.
(32) *Mémoires et Compte Rendu des Travaux*, Soc. Ing. Civ. de France, Paris, France.
(33) *Le Génie Civil*, Paris, France.
(34) *Portefeuille Economiques des Machines*, Paris, France.
(35) *Nouvelles Annales de la Construction*, Paris, France.
(37) *Revue de Mécanique*, Paris, France.
(38) *Revue Générale des Chemins de Fer et des Tramways*, Paris, France.
(41) *Modern Machinery*, Chicago, Ill., 10c.
(42) *Proceedings*, Am. Inst. Elec. Engrs., New York City, 50c.
(43) *Annales des Ponts et Chaussées*, Paris, France.
(44) *Journal*, Military Service Institution, Governors Island, New York Harbor, 50c.
(45) *Mines and Minerals*, Scranton, Pa., 20c.
(46) *Scientific American*, New York City, 8c.
(47) *Mechanical Engineer*, Manchester, England.
(48) *Zeitschrift*, Verein Deutscher Ingenieure, Berlin, Germany.
(49) *Zeitschrift für Bauwesen*, Berlin, Germany.
(50) *Stahl und Eisen*, Düsseldorf, Germany.
(51) *Deutsche Bauzeitung*, Berlin, Germany.
(52) *Rigasche Industrie-Zeitung*, Riga, Russia.
(53) *Zeitschrift*, Oesterreichischer Ingenieur und Architekten Verein, Vienna, Austria.
(54) *Transactions*, Am. Soc. C. E., New York City, $5.
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Proper Treatment of Boiler Feed Water.* A. J. Boardman. (64) Mar. 23.
Tests of Soil for Use in Constructing an Earth Dam for a Water-Works Reservoir, Baltimore, Md. Emory Sudler. (13) Mar. 25.
A Reinforced-Concrete Pressure Sedimentation Tank.* Harold D. Elfreth. (13) Mar. 25.
The Hydro-Electric Plant of the Connecticut River Power Company.* (14) Mar. 27.
The Chuviscar Dam of the Chihuahua Water-Works.* (14) Mar. 27.
Roosevelt Tunnel.* R. L. Herrick. (45) Apr.
The Undermining of a Reinforced Concrete Dam at Pittsfield, Mass.* (14) Apr. 1.
A New Water-Supply for the City of Vancouver, B. C.* H. M. Burwell. (13) Apr. 1.
The Construction of the Plant of the Connecticut River Power Company.* George S. Hewins. (14) Apr. 3.
Development of the Rocky Creek Station of the Southern Power Company.* C. A. Mees, M. Am. Soc. C. E. (14) Apr. 3.
Constructing a Line of Steel Pipe 12½ Feet in Diameter.* (14) Apr. 3.
Construction Work on the Los Angeles Aqueduct.* E. W. Bannister. (14) Apr. 3.
Hydraulic Dam Construction on the Conconully Dam, Washington.* D. C. Henny. (14) Apr. 3.
The La Prele Dam.* (14) Apr. 3.
The Design of Intakes for Hydro-Electric Plants.* Jean Bart. Balcomb, Assoc. Am. Inst. E. E. (13) Apr. 8.
The Construction of the Submerged Intake of the Gary, Indiana, Water-Works.* (14) Apr. 10.
The Compressed Air Plant for the Rondout Siphon.* (14) Apr. 10.
Note sur la Construction de Trois Barrages-Réservoirs dans la Vallée de l'Ondaine (Loire). Reuss. (43) Nov.

Waterway.

Notes upon Docks and Harbors.* Luther Wagoner, M. Am. Soc. C. E. (54) Vol. 62.
Concrete Piles Used in the Steamship Terminals at Brunswick, Ga., and in Navy Yard Pier at Charleston, S. C.* M. M. Cannon, M. Am. Soc. C. E. (Paper read before the Bost. Soc. Civ. Engrs.) (1) Jan.
Caving Banks. B. M. Harrod. (Paper read before the Louisiana Eng. Soc.) (1) Jan.
The Water Hyacinth Problem.* John Klorer. (Paper read before the Louisiana Eng. Soc.) (1) Jan.
Improvement on the Upper Mississippi River.* C. McD. Townsend, M. W. S. E. (4) Feb.
High Tides of the Bay of Fundy.* J. W. Ridpath. (3) Mar.
Barriers to International Trade; a Study of the Problems at the Ports of Liverpool and New York.* Lewis M. Haupt. (10) Serial beginning Apr.
Concerning the Gatun Dam and Earth Dams in General.* (13) Apr. 1.
The Black Rock Lock Cofferdam.* (14) Apr. 3.
A Large Concrete Retaining Wall.* (14) Apr. 3.
Progress on the Improvement of the Detroit River.* (14) Apr. 3.
The Improvement of the Southwest Pass at the Mouth of the Mississippi River.* (14) Apr. 3.
Construction Work on the New York State Barge Canal.* (14) Apr. 3.
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Canal de Berry; Réparation du Pont-Canal de la Tranchasse.* Barrat. (43) Nov.

*Illustrated.

PROCEEDINGS

OF THE

AMERICAN SOCIETY

OF

CIVIL ENGINEERS

(INSTITUTED 1852)

VOL. XXXV—No. 5.

MAY, 1909.

Edited by the Secretary, under the direction of the Committee on Publications.

Reprints from this publication, which is copyrighted, may be made on condition that the full title of Paper, name of Author, page reference, and date of presentation to the Society, are given.

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NEW YORK 1909

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American Society of Civil Engineers

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ON STATUS OF METRIC SYSTEM: Stacy B. Opdyke, Jr., D. A. Molitor.

ON ENGINEERING EDUCATION: Desmond FitzGerald, Benjamin M. Harrod, Onward Bates, D. W. Mead, Charles Hansel.

ON STEEL COLUMNS AND STRUTS: Austin L. Bowman, Alfred P. Boller, Emil Gerber, Charles F. Loweth, Ralph Modjeski, Frank C. Osborn, George H. Pegram, Lewis D. Rights, George F. Swain, Emil Swensson, Joseph R. Worcester.

ON BITUMINOUS MATERIALS FOR ROAD CONSTRUCTION: W. W. Crosby, A. W. Dean, H. K. Bishop, A. H. Blanchard.

The House of the Society is open from 9 A.M. to 10 P.M. every day, except Sundays, Fourth of July, Thanksgiving Day and Christmas Day.

HOUSE OF THE SOCIETY—220 WEST FIFTY-SEVENTH STREET, NEW YORK.

TELEPHONE NUMBER.....5913 Columbus.
CABLE ADDRESS....."Ceas, New York."

Vol. XXXV. MAY, 1909. No. 5.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PROCEEDINGS

This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

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MINUTES OF MEETINGS

OF THE SOCIETY

April 21st, 1909.—The meeting was called to order at 8.30 P. M.; James Owen, M. Am. Soc. C. E., in the chair; Chas. Warren Hunt, Secretary; and present, also, 108 members and 53 guests.

A paper by C. S. Bissell, M. Am. Soc. C. E., entitled "The Maximum Weights of Slow Freight Trains," was presented by the Secretary, who also read a communication on the subject from Mr. L. A. Riley, 2d.

A paper by W. W. Crosby, M. Am. Soc. C. E., entitled "Sampittic Surfacing," was presented by the author and discussed orally by Messrs. A. H. Blanchard, Francis T. Smith, Philip P. Sharples, H. W. Clark, J. W. Howard, and the author. Written communications on the subject from Messrs. L. W. Page and Prevost Hubbard were presented by the Secretary.

W. W. Crosby, M. Am. Soc. C. E., offered the following resolution:

"*Resolved:* That the Board of Direction of the American Society

of Civil Engineers be requested to appoint a Special Committee to Report on Bituminous Materials for Road Construction, and on Standards for Their Test and Use."

The Resolution, being duly seconded, was adopted by a vote of more than 25 Corporate Members.

The Secretary announced the following deaths:

LEWIS FREDERICK RICE, elected Member January 2d, 1889; died April 13th, 1909.

ANDREW ROSEWATER, elected Member October 3d, 1883; died April 17th, 1909.

AUGUST MAYER, elected Associate Member June 1st, 1892; died March 25th, 1909.

Adjourned.

May 5th, 1909.—The meeting was called to order at 8.30 P. M.; President Onward Bates in the chair; Chas. Warren Hunt, Secretary; and present, also, 132 members and 31 guests.

The minutes of the meetings of March 17th and April 7th, 1909, were approved as printed in the *Proceedings* for April, 1909.

The Secretary announced that in accordance with the Resolution adopted by the Society at its meeting on April 21st, 1909, the Board of Direction, at its regular meeting, May 4th, 1909, had appointed the following Special Committee to Report on Bituminous Materials for Road Construction, and on Standards for Their Test and Use: Messrs. W. W. Crosby, A. W. Dean, H. K. Bishop, and A. H. Blanchard.

A paper by Myron S. Falk, M. Am. Soc. C. E., entitled "A System of Cost Keeping," was presented by the author and discussed orally by Messrs. William M. Black, Robert B. Stanton, T. Kennard Thomson, Willard D. Lockwood, William B. Reed, and Guy W. Culgin.

Written communications on the subject from Messrs. P. L. Reed, W. C. Hammatt, and Emile Low, were read by the Secretary.

A paper by C. W. Birch-Nord, Assoc. M. Am. Soc. C. E., entitled "The Design of Elevated Tanks and Stand-Pipes," was presented by the Secretary, who also read communications on the subject from Messrs. Charles B. Burdick, and O. E. Selby. The paper was discussed orally by Messrs. William M. Black, and James M. Porter.

The Secretary announced the election of the following candidates by the Board of Direction on April 6th, 1909:

AS MEMBERS.

JOHN BENNINGTON BERRY, Chicago, Ill.
OSCAR SIDNEY BOWEN, Spokane, Wash.
WILLIAM HOWELL CALDWELL, Tampa, Fla.
JOSEPH KITTREDGE CHOATE, Cooperstown, N. Y.

THOMAS CURTIS CLARKE, New York City.
CHARLES LEO CRANDALL, Galveston, Tex.
HARRY HAWGOOD, Los Angeles, Cal.
THOMAS NOTTINGHAM JACOB, Herrin, Ill.
WILLIAM PRENTISS MORSE, Newton, Mass.
WILLIAM RODNEY PATTERSON, Chicago, Ill.
HENRY OTIS POND, Tenafly, N. J.
WALTER JUSTIN SHERMAN, Toledo, Ohio.
WILLIS ROBINSON TENNEY, Brooklyn, N. Y.
EDWIN DERICKSON VINCENT, Yuma, Ariz.

AS ASSOCIATE MEMBERS.

JOHN HOWARD ALEXANDER, St. Louis, Mo.
WALTER CHARNLEY, Rio Grande do Sul, Brazil.
FRANK LEMUEL CLAPP, Cornwall-on-Hudson, N. Y.
JOHN SEBASTIAN CONWAY, Portland, Ore.
BERTRAM EDWIN CORLETT, Seattle, Wash.
CHESTER CAWTHORNE DERBY, Richmond Hill, N. Y.
ARCHIBALD MARVINE GILBERT, Twin Falls, Idaho.
WILLIAM STEWART GREENE, New Brunswick, N. J.
RUSSELL THEODORE HARTMAN, Des Moines, Iowa.
WILLIAM SYLVESTER HAZELTON, Detroit, Mich.
LARS RASMUS JORGENSEN, San Francisco, Cal.
ALFRED BROUGHTON KISSACK, Leavenworth, Kans.
WILLIAM ARTHUR LAFLER, Rome, N. Y.
CLARENCE BOOTH LAMONT, Seattle, Wash.
WALTER FRANKLIN LINEBERGER, Torreón, Coahuila, Mexico.
THOMAS CLAWSON MARSHALL, Middletown, N. Y.
BERTRAND CLIFFORD MARTIN, Watertown, N. Y.
FRANK DANA NASH, Manila, Philippine Islands.
HARRY BRADFORD POND, New York City.
HARRY IVES SHOEMAKER, Puunene, Hawaii.

AS ASSOCIATES.

JOHN COFFEE HAYS, Visalia, Cal.
FRANK MAYHEW TALBOT, Glen Ridge, N. J.

AS JUNIORS.

CHESTER DANIEL ASHBROOK, Seattle, Wash.
HARRY MONTIFIORE BERGMAN, New York City.
LYMAN EDGAR BISHOP, Denver, Colo.
MYRON CARLOS BURR, Vancouver, B. C., Canada.
STANLEY GARDNER CUTLER, Manila, Philippine Islands.
FRANK JOHN FELGENHAUER, Brooklyn, N. Y.

Robert Maximilian Feustel, Madison, Wis.
Alvero Charles Gregson, New York City.
Charles Burridge Hawley, Cleveland, Ohio.
Cyrus Pierce Howes, St. Louis, Mo.
Andrew Peach Rollins, Eagle Lake, Tex.
Harvey Hillyer Sims, Savannah, Ga.
Thomas Harrison Winchester, Macon, Ga.

The Secretary announced the transfer of the following candidates by the Board of Direction on May 4th, 1909:

From Associate Member to Member.

Philip Asfordby Beatty, Baltimore, Md.
Arthur Horace Blanchard, Providence, R. I.
Clarence Austin Crane, New York City.
Frank Tenney Darrow, Lincoln, Nebr.
Leonard Henry Davis, Sault Ste. Marie, Ont., Canada.
Frank Lawton Getman, Havana, Cuba.
Harry Thomas Paterson, Beaufort, N. C.
Samuel Payson Waldron, East Orange, N. J.

From Junior to Associate Member.

William Burton, Detroit, Mich.
William Hale Phillips, Spokane, Wash.
Walter John Plogsted, New York City.
William Purves Taylor, Philadelphia, Pa.

The Secretary announced the following deaths:

Amzi Lorenzo Barber, elected Fellow March 19th, 1886; died April 18th, 1909.

John Arnold Ubsdell, elected Associate June 7th, 1905; died April 23d, 1909.

Mace Moulton, elected Member June 4th, 1884; died April 27th, 1909.

Adjourned.

OF THE BOARD OF DIRECTION

(Abstract)

May 4th, 1909.—President Bates in the chair; Chas. Warren Hunt, Secretary; and present, also, Messrs. Andrews, Christie, Churchill, Harrison, Horton, Kittredge, Knap, Macdonald, Noble, Pegram, Schneider, Stearns, Stuart, Thompson, and Tillson.

Ballots for Membership were canvassed, and 14 Members, 20 Associate Members, 2 Associates, and 13 Juniors were elected, and 4 Juniors were transferred to the grade of Associate Member.

Eight Associate Members were transferred to the grade of Member.

The following resolution was adopted:*

"*Resolved,* That a Special Committee be appointed to Report on Bituminous Materials for Road Construction, and on Standards for their Test and Use, to consist of Messrs. W. W. Crosby, A. W. Dean, H. K. Bishop, and A. H. Blanchard; this Committee to be added to later if the Board deems it necessary."

The Secretary reported the completion of the new tier of book stacks for the Library.

Applications were considered and other routine business transacted.

Adjourned.

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* See page 232 for resolution adopted by the Society.

ANNOUNCEMENTS

The House of the Society is open from 9 A. M. to 10 P. M., every day, except Sundays, Fourth of July, Thanksgiving Day, and Christmas Day.

MEETINGS

June 2d, 1909.—8.30 P. M.—Two papers will be presented for discussion as follows: "Tests of Built-Up Steel and Wrought-Iron Compression Pieces," by Arthur N. Talbot, M. Am. Soc. C. E., and Herbert F. Moore, Esq.; and "Caisson Disease and Its Prevention," by Henry Japp, M. Am. Soc. C. E.

These papers were printed in *Proceedings* for April, 1909.

September 1st, 1909.—8.30 P. M.—At this meeting two papers will be presented for discussion, as follows: "Concrete Piles," by Howard J. Cole, M. Am. Soc. C. E., and "Hydro-Electric Power in Canada," by Cecil B. Smith, M. Am. Soc. C. E.

These papers are printed in this number of *Proceedings*.

ANNUAL CONVENTION

The Forty-first Annual Convention of the Society will be held at the Mount Washington Hotel, Bretton Woods, N. H., from July 6th to July 9th, 1909, inclusive.

The general arrangements for the Convention are in the hands of the following Committees:

COMMITTEE OF THE BOARD OF DIRECTION

F. W. Hodgdon,

G. W. Tillson, Chas. Warren Hunt.

LOCAL COMMITTEE

J. W. Ellis,

A. W. Dean, J. F. Stevens,
H. W. Hayes, S. E. Tinkham,
George A. Kimball, H. D. Woods.

A circular will be issued about June 1st, 1909, giving information relative to transportation, and further details of the programme.

PAPERS AND DISCUSSIONS

The first volume of *Transactions* for 1909 (Vol. LXII) has been issued. There will be three additional volumes issued during the year.

It is hoped that members and others who take part in the discussion of the papers presented will revise their remarks promptly, and that all written communications from those who cannot attend the meetings will be sent in at the earliest possible date after the issue of a paper in *Proceedings*. The issue of volumes of *Transactions* is

dependent on the closing of discussions, and the co-operation of the membership will now be more necessary in this matter than heretofore, because four volumes are to be issued during the year, instead of two, and, to accomplish this, a definite date of issue for each has been established. It is expected that the second volume of 1909 will be issued on June 30th, and the third and fourth on September 30th and December 31st, respectively.

All papers accepted by the Publication Committee are classified by the Committee with respect to their availability for discussion at meetings.

Papers, which, from their general nature, appear to be of a character suitable for oral discussion, will be published as heretofore in *Proceedings*, and set down for presentation to a future meeting of the Society, and, on these, oral discussion, as well as written communications, will be solicited.

All papers which do not come under this heading, that is to say, those which, from their mathematical or technical nature, in the opinion of the Committee, are not adapted to oral discussion, will not be scheduled for presentation to any meeting. Such papers will be published in *Proceedings* in the same manner as those which are to be presented at meetings, but written discussions, only, will be requested for subsequent publication in *Proceedings* and with the paper in the volumes of *Transactions*.

PRIVILEGES OF ENGINEERING SOCIETIES EXTENDED TO MEMBERS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

Members of the American Society of Civil Engineers will be welcomed by the following Engineering Societies, both to the use of their Reading Rooms and at all Meetings:

American Institute of Mining Engineers, 29 West Thirty-ninth Street, New York City.

Associação dos Engenheiros Civis Portuguezes, Lisbon, Portugal.

Australasian Institute of Mining Engineers, Melbourne, Victoria, Australia.

Boston Society of Civil Engineers, 715 Tremont Temple, Boston, Mass.

Brooklyn Engineers' Club, 197 Montague Street, Brooklyn, N. Y.

Canadian Society of Civil Engineers, 413 Dorchester Street, West, Montreal, Que., Canada.

Civil Engineers' Society of St. Paul, St. Paul, Minn.

Cleveland Engineering Society, 718 Caxton Building, Cleveland, Ohio.

Cleveland Institute of Engineers, Middlesbrough, England.

Colorado Association of Members, Am. Soc. C. E., H. J. Burt, Secy., 235 Equitable Building, Denver, Colo.

Engineers' and Architects' Club of Louisville, Ky., 303 Norton Building, Fourth and Jefferson Streets, Louisville, Ky.

Engineers' Club of Baltimore, Baltimore, Md.

Engineers' Club of Central Pennsylvania, Corner Second and Walnut Streets, Harrisburg, Pa.

Engineers' Club of Minneapolis, 17 South Sixth Street, Minneapolis, Minn.

Engineers' Club of Philadelphia, 1317 Spruce Street, Philadelphia, Pa.

Engineers' Club of St. Louis, 3817 Olive Street, St. Louis, Mo.

Engineers' Club of Toronto, 96 King Street, West, Toronto, Ont., Canada.

Engineers' Society of Western Pennsylvania, 803 Fulton Building, Pittsburg, Pa.

Institute of Marine Engineers, 58 Romford Road, Stratford, London, E., England.

Institution of Engineers of the River Plate, Buenos Aires, Argentine Republic.

Institution of Naval Architects, 5 Adelphi Terrace, London, W. C., England.

Junior Institution of Engineers, 39 Victoria Street, Westminster, S. W., London, England.

Koninklijk Instituut van Ingenieurs, The Hague, The Netherlands.

Louisiana Engineering Society, 604 Tulane-Newcomb Building, New Orleans, La.

Memphis Engineering Society, Memphis, Tenn.

Midland Institute of Mining, Civil and Mechanical Engineers, Sheffield, England.

Montana Society of Engineers, Butte, Montana.

North of England Institute of Mining and Mechanical Engineers, Newcastle-upon-Tyne, England.

Oesterreichischer Ingenieur- und Architekten-Verein, Eschenbachgasse 9, Vienna, Austria.

Pacific Northwest Society of Engineers, 617-618 Pioneer Building, Seattle, Wash.

Rochester Engineering Society, Rochester, N. Y.

Sachsischer Ingenieur- und Architekten-Verein, Dresden, Germany.

Sociedad Colombiana de Ingenieros, Bogota, Colombia.

Societe des Ingenieurs Civils de France, 19 Rue Blanche, Paris, France.

Society of Engineers, 17 Victoria Street, Westminster, S. W., London, England.

Svenska Teknologforeningen, Brunkebergstorg 18, Stockholm, Sweden.

Tekniske Forening, Vestre Boulevard 18–1, Copenhagen, Denmark.

Western Society of Engineers, 1737 Monadnock Block, Chicago, Ill.

SEARCHES IN THE LIBRARY

In January, 1902, the Secretary was authorized to make searches in the Library, upon request, and to charge therefor the actual cost to the Society for the extra work required. Since that time many searches have been made, and bibliographies and other information on special subjects furnished.

The resulting satisfaction, to the members, who have made use of the resources of the Society in this manner, has been expressed frequently, and leaves little doubt that, if it were generally known to the membership that such work would be undertaken, many would avail themselves of it.

The cost is trifling compared with the value of the time of an engineer who looks up such matters himself, and the work can be performed quite as well, and much more quickly, by persons familiar with the Library.

In asking that such work be undertaken, members should specify clearly the subject to be covered, and whether references to general books only are desired, or whether a complete bibliography, involving search through periodical literature, is desired.

In reference to this work, the Appendix* to the Annual Report of the Board of Direction for the year ending December 31st, 1906, contains a summary of all searches made to that date.

* *Proceedings*, Vol. XXXIII, p. 20 (January, 1907).

ACCESSIONS TO THE LIBRARY

(From April 13th to May 10th, 1909)

DONATIONS*

SANITATION AND SANITARY ENGINEERING.

By Wm. Paul Gerhard. Second Revised and Enlarged Edition of "Sanitary Engineering." Cloth, 7½ x 5 in., 6 + 175 pp. New York, The Author, 1909. $1.50 net.

In this edition of "Sanitary Engineering" the title has been changed to "Sanitation and Sanitary Engineering," and the work enlarged by additional matter to more than twice its original size. After defining the term, sanitary engineering, the author describes briefly the course of study and special training necessary to the practice of this branch of the profession, including the courses given at the Massachusetts Institute of Technology and Cornell and Columbia Universities. This is followed by a general and condensed review of the various kinds of work and problems with which the sanitary engineer has to deal. The Contents are: Sanitary Engineering—The Profession and Its Practice; The Work of the Sanitary Engineer in Time of Epidemics, in Time of War, and in Sudden Calamities in Civic Life; A Half Century of Sanitation (1850-1900); Sanitation in Greater New York: Sanitation in Russia; Reference to Other Works of the Author Dealing More in Detail With Problems of Sanitary Engineering.

BEITRAG ZUR BERECHNUNG UND AUSFÜHRUNG DER STAUMAUERN.

Von Franz Kreuter. Cloth, 8½ x 5½ in., illus., 54 pp. München und Berlin, R. Oldenbourg, 1909. 2 marks, 40 pfennigs.

The author, in his prefatory remarks, states that this work constitutes an abstract of his lectures at the Münich Technische Hochschule, and is the result of many years' work. Other investigators have pointed out the defects in the old methods of calculating masonry dams, he says, but have put forward no new method, while he has fulfilled the new demands and found a solution of the problem which he offers to the profession. He claims, also, that while the present work is more complete and more accurate than his former treatise on the same subject, it is, at the same time, simpler and more comprehensive. The Contents are: Praktische Vorbedingungen; Theoretische Vorbedingungen; Entwicklung der Bestimmungsgleichungen; Anwendung; Schlussbemerkungen.

HANDBOOK OF TIMBER PRESERVATION.

By Samuel M. Rowe, M. Am. Soc. C. E. Souvenir Edition, Revised. Leather, 6 x 4 in., illus., 402 pp. Chicago, Pettibone, Sawtelle & Co., 1909. $5. (Presented by the Author.)

This edition, which the author states in a "valedictory" to be the last, is enlarged by about seventy-five pages of new matter, the first part being practically unchanged. The additional matter includes tables on absorbent properties of timber, reprints of letters to the *Engineering News*, several pages on fungus culture, with plans for culture room, records of tie renewals, notes on the open-tank method of treatment, etc.

MINING METHODS IN EUROPE.

Written After a Visit to Many of the Metal and Coal Mines of Great Britain, France and Germany. By Lucius W. Mayer. Cloth. 9½ x 6 in., illus., 9 + 169 pp. New York and London, Hill Publishing Company, 1909. $2.50 net.

This volume describes what the author characterizes as the more or less distinct and often extraordinary systems of mining practised in various countries of Europe. The preface speaks of the scarcity of mining literature, especially on the subject of the method of underground attack, and states that this work is intended to help fill this lack, this feature, in every case, being explained in detail. In Part I, Mining in Great Britain, the chapter headings are: Mining in Great

*Unless otherwise specified, books in this list have been donated by the publisher.

Britain; Parkside Mine, Cumberland; Creator Moor Mine, Cumberland; Ullcoat Mine, Cumberland; Hodbarrow Mine, Cumberland; Longwall Methods of Mining Flat Seams; Seaton-Delaval Colliery, Northumberland; Chopwell Colliery, Northumberland; Kimblesworth Colliery, Northumberland; Watnall Colliery, Midland District; Coal Mining in South Staffordshire; Spawood Mine, Yorkshire; Park Pit, Yorkshire; Oakley Slate Quarries, North Wales. In Part II, Mining in Germany, France and Other Countries, the chapter headings are: Lead Mining in Mechernich, Prussia; The Flushing System; Coal Mining in Silesia; Flushing System at Gute Hoffnung Mine, Germany; Ilsede Hütte Mine, Peine, Germany; Coal Mines of Carmaux, France; Notes on Mining, Timbering, and Lighting.

TRIGONOMETRISCHE LÄNGENBESTIMMUNG GEODÄTISCHER GRUNDLINIEN.

Von A. Tichy. Paper, 11 x 7½ in., illus., 67 pp. Wien, Der Oesterreichische Ingenieure-und Architekte-Verein, 1909.

RAILROAD STRUCTURES AND ESTIMATES.

By J. W. Orrock. Cloth, 9 x 6 in., illus., 6 + 270 pp. New York, John Wiley & Sons; London, Chapman & Hall, Limited, 1909. $3 net.

The intention of this work, as set forth in the preface, is to cover in a brief and concise form the numerous subjects which enter into the engineer's estimates of railroad building, for the purpose of ready reference as to general construction and cost, on a business rather than on a technical basis. It is also stated that data to suit all conditions being impossible, the weights, quantities, and costs are given in detail in most instances. In many of the tables blank spaces are left for the addition by the user of the work of his own prices and records. The Contents are: Track Material; Fences, Gates, Sign Posts, Road Crossings and Guards; Culverts; Bridges; Buildings; Water Stations; Shops; Specifications and Contracts; Estimating Notes; Index.

Gifts have also been received from the following:

Allen, W. F. 1 pam.
Am. Electrochemical Soc. 1 vol.
Am. Iron and Steel Assoc. 1 pam.
Am. Min. Congress. 1 vol.
Am. Ry. Eng. and M. of W. Assoc. 2 pam.
Am. Soc. of Municipal Impvts. 1 bound vol.
Bandy, James M. 2 pam.
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Handbuch der Ingenieurwissenschaften. Vol. 3, Part IV. Die Baumaschinen. Von F. Linke. Zweite vermehrte Auflage. Wilhelm Engelmann, Leipzig, 1908.

Disposal of Liquid Refuse from Distilleries. Sixth Report of the Commissioners Appointed to Inquire and Report What Methods of Treating and Disposing of Sewage (Including any Liquid from any Factory or Manufacturing Process) May Properly be Adopted. Wyman & Sons, Ltd., London, 1909.

San Diego, a Comprehensive Plan for Its Improvement. By John Nolen. Geo. H. Ellis Co., Boston, 1908.

The Gas Engine. By Forrest R. Jones. John Wiley & Sons, New York; Chapman & Hall, Ltd., London, 1909.

Hydraulique Fluviale. La Forme du Lit des Rivières à Fond Mobile. Par L. Fargue. Encyclopédie des Travaux Publics. Gauthier-Villars, Paris, 1908.

An Elementary Manual of Radiotelegraphy and Radiotelephony for Students and Operators. By J. A. Fleming. Longmans, Green & Co., London, 1908.

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	M.	May 4, 1909
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	Assoc. M.	April 5, 1905
	M.	May 4, 1909
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MARSH, EMMETT LINCOLN. Asst. Engr., S. P. Co., Box 21, Bowman, Cal.

MILLER, HENRY LANARK. 175 Calle 5 Pueblo General Paz, Cordoba, Argentine Republic.

PARSONS, AUGUSTUS TABER. Bank of Bakersfield Bldg., Bakersfield, Cal.

RIEGEL, ROSS MILTON. Bruceton Mills, Preston Co., W. Va.

ROBBINS, HALLET RICE. P. O. Box 51, Seattle, Wash.

RYAN, RICHARD R. 230 N. 3d St., Phœnix, Ariz.

RYAN, WALTER J. Res. Engr., N. P. Ry., Cannon Ball, N. Dak.

SCHMID, FRANCIS RAUCH. 30 Wall St., Bethlehem, Pa.

SHAFER, JAMES CHARLES FORSYTHE. 140 West 48th St., New York City.

SMITH, FORD CUSHING. Care, Rev. G. H. Smith, Rhinebeck, N. Y.

SWETT, EVERETT HAROLD. Room 411, Temple Court Bldg., Atlanta, Ga.

VOGEL, JOHN LEONARD. 61 Belmont Ave., Jersey City, N. J.

WARLOW, ADONIRAM JUDSON. 1718 S. 18th St., Philadelphia, Pa.

WILCOX, FRANK LESLIE. 515 West 111th St., New York City.

WITTSTEIN, HERMAN LEWIS. Care, Quick's Rock Cliff House, High Falls, Ulster Co., N. Y.

DEATHS.

BARBER, AMZI LORENZO. Elected Fellow, March 19th, 1886; died April 18th, 1909.

MAYER, AUGUST. Elected Associate Member, June 1st, 1892; died March 25th, 1909.

MOULTON, MACE. Elected Member, June 4th, 1884; died April 27th, 1909.

RICE, LEWIS FREDERICK. Elected Member, January 2d, 1889; died April 13th, 1909.

ROSEWATER, ANDREW. Elected Member, October 3d, 1883; died April 17th, 1909.

UBSDELL, JOHN ARNOLD. Elected Associate, June 7th, 1905; died April 23d, 1909.

MONTHLY LIST OF RECENT ENGINEERING ARTICLES OF INTEREST

(April 13th to May 10th, 1909)

NOTE.—*This list is published for the purpose of placing before the members of the Society, the titles of current engineering articles, which can be referred to in any available engineering library, or can be procured by addressing the publication directly, the address and price being given wherever possible.*

LIST OF PUBLICATIONS

In the subjoined list of articles, references are given by the number prefixed to each journal in this list.

(1) *Journal*, Assoc. Eng. Soc., 31 Milk St., Boston, Mass., 30c.
(2) *Proceedings*, Engrs. Club of Phila., 1317 Spruce St., Philadelphia, Pa.
(3) *Journal*, Franklin Inst., Philadelphia, Pa., 50c.
(4) *Journal*, Western Soc. of Engrs., Monadnock Blk., Chicago, Ill.
(5) *Transactions*, Can. Soc. C. E., Montreal, Que., Canada.
(6) *School of Mines Quarterly*, Columbia Univ., New York City, 50c.
(7) *Technology Quarterly*, Mass. Inst. Tech., Boston, Mass., 75c.
(8) *Stevens Institute Indicator*, Stevens Inst., Hoboken, N. J., 50c.
(9) *Engineering Magazine*, New York City, 25c.
(10) *Cassier's Magazine*, New York City, 25c.
(11) *Engineering* (London), W. H. Wiley, New York City, 25c.
(12) *The Engineer* (London), International News Co., New York City, 35c.
(13) *Engineering News*, New York City, 15c.
(14) *The Engineering Record*, New York City, 12c.
(15) *Railroad Age Gazette*, New York City, 15c.
(16) *Engineering and Mining Journal*, New York City, 15c.
(17) *Electric Railway Journal*, New York City, 10c.
(18) *Railway and Engineering Review*, Chicago, Ill., 10c.
(19) *Scientific American Supplement*, New York City, 10c.
(20) *Iron Age*, New York City, 10c.
(21) *Railway Engineer*, London, England, 25c.
(22) *Iron and Coal Trades Review*, London, England. 25c.
(23) *Bulletin*, American Iron and Steel Assoc., Philadelphia, Pa.
(24) *American Gas Light Journal*, New York City, 10c.
(25) *American Engineer*, New York City, 20c.
(26) *Electrical Review*, London, England.
(27) *Electrical World*, New York City, 10c.
(28) *Journal*, New England Water-Works Assoc., Boston, Mass., $1.
(29) *Journal*, Royal Society of Arts, London, England, 15c.
(30) *Annales des Travaux Publics de Belgique*, Brussels, Belgium.
(31) *Annales* de l'Assoc. des Ing. Sortis des Ecoles Spéciales de Gand, Brussels, Belgium.
(32) *Mémoires et Compte Rendu des Travaux*, Soc. Ing. Civ. de France, Paris, France.
(33) *Le Génie Civil*, Paris, France.
(34) *Portefeuille Economiques des Machines*, Paris, France.
(35) *Nouvelles Annales de la Construction*, Paris, France.
(37) *Revue de Mécanique*, Paris, France.
(38) *Revue Générale des Chemins de Fer et des Tramways*, Paris, France.
(41) *Modern Machinery*, Chicago, Ill., 10c.
(42) *Proceedings*, Am. Inst. Elec. Engrs., New York City, 50c.
(43) *Annales des Ponts et Chaussées*, Paris, France.
(44) *Journal*, Military Service Institution, Governors Island, New York Harbor, 50c.
(45) *Mines and Minerals*, Scranton, Pa., 20c.
(46) *Scientific American*, New York City, 8c.
(47) *Mechanical Engineer*, Manchester, England.
(48) *Zeitschrift*, Verein Deutscher Ingenieure, Berlin, Germany.
(49) *Zeitschrift für Bauwesen*, Berlin, Germany.
(50) *Stahl und Eisen*, Düsseldorf, Germany.
(51) *Deutsche Bauzeitung*, Berlin, Germany.
(52) *Rigasche Industrie-Zeitung*, Riga, Russia.
(53) *Zeitschrift*, Oesterreichischer Ingenieur und Architekten Verein, Vienna, Austria.
(54) *Transactions*, Am. Soc. C. E., New York City, $5.
(55) *Transactions*, Am. Soc. M. E., New York City, $10.
(56) *Transactions*, Am. Inst. Min. Engrs., New York City, $5.

(57) *Colliery Guardian*, London, England.
(58) *Proceedings*, Eng. Soc. W. Pa., 803 Fulton Bldg., Pittsburg, Pa., 50c.
(59) *Transactions*, Mining Inst. of Scotland, London and Newcastle-upon-Tyne, England.
(60) *Municipal Engineering*, Indianapolis, Ind., 25c.
(61) *Proceedings*, Western Railway Club, 225 Dearborn St., Chicago, Ill., 25c.
(62) *Industrial World*, 59 Ninth St., Pittsburg, Pa.
(63) *Minutes of Proceedings*, Inst. C. E., London, England.
(64) *Power*, New York City, 20c.
(65) *Official Proceedings*, New York Railroad Club, Brooklyn, N. Y., 15c.
(66) *Journal of Gas Lighting*, London, England, 15c.
(67) *Cement and Engineering News*, Chicago, Ill., 25c.
(68) *Mining Journal*, London, England.
(70) *Engineering Review*, New York City, 10c.
(71) *Journal*, Iron and Steel Inst., London, England.
(73) *Electrician*, London, England, 18c.
(74) *Transactions*, Inst. of Min. and Metal., London, England.
(75) *Proceedings*, Inst. of Mech. Engrs., London, England.
(76) *Brick*, Chicago, Ill., 10c.
(77) *Journal*, Inst. Elec. Engrs., London, England.
(78) *Beton und Eisen*, Vienna, Austria.
(79) *Forscherarbeiten*, Vienna, Austria.
(80) *Tonindustrie Zeitung*, Berlin, Germany.
(81) *Zeitschrift für Architektur und Ingenieurwesen*, Wiesbaden, Germany.
(82) *Dinglers Polytechnisches Journal*, Berlin, Germany.
(83) *Progressive Age*, New York City, 15c.
(84) *Le Ciment*, Paris, France.
(85) *Proceedings*, Am. Ry. Eng. and M. of W. Assoc., Chicago, Ill.
(86) *Engineering-Contracting*, Chicago, Ill., 10c.
(87) *Roadmaster and Foreman*, Chicago, Ill., 10c.
(88) *Bulletin* of the International Ry. Congress Assoc., Brussels, Belgium.
(89) *Proceedings*, Am. Soc. for Testing Materials, Philadelphia, Pa.
(90) *Transactions*, Inst. of Naval Archts., London, England.
(91) *Transactions*, Soc. Naval Archts. and Marine Engrs., New York City.
(92) *Bulletin*, Soc. d'Encouragement pour l'Industrie Nationale, Paris, France.
(93) *Revue de Métallurgie*, Paris, France, 4 fr. 50.
(94) *The Boiler Maker*, New York City, 10c.
(95) *International Marine Engineering*, New York City, 20c.

LIST OF ARTICLES.

Bridge.

Nickel Steel for Bridges.* J. A. L. Waddell, M. Am. Soc. C. E. **(54)** Vol. 63.
Bridge Designing.* Carl Gayler. (Paper read before the Engrs.' Club of St. Louis). **(1)** Mar.
Annual Address (Engineering Construction in Montana). Archer E. Wheeler. (Paper read before the Montana Soc. of Engrs.) **(1)** Mar.
A New Arch Curve, the Parabolic Oval.* Charles Worthington, M. Am. Soc. C. E. **(13)** Apr. 15.
The Grand River Bridge of the Lake Shore & Michigan Southern Railway.* **(14)** Apr. 24.
Methods of Field Work in Connection with Caisson Sinking. B. Van De Greyn. (From *Technograph.*) **(86)** Apr. 28.
The Missouri River and its Bridges.* J. Y. Oleson. **(13)** Apr. 29.
Concrete in Arched Bridge Construction. E. P. Wells. (Paper read before the Concrete Inst.) **(21)** May.
The Construction of the Grand River Bridge.* **(14)** May 1.
Remarques sur la Stabilités des Viaducs.* Bonneau. **(43)** Jan.
Le Pont Doumer de 1 680 Mètres d'Ouverture sur le Fleuve Rouge, à Hanoï (Tonkin).* **(33)** Apr. 3.
Die Verbreiterung der Wilhelms-Brücke in Frankfort a. M.* Kölle. (Paper read before the Deutsche Beton Verein zu Berlin.) **(51)** Apr. 14.
Belastungsprobe der Beton-Gelenkbrücke der Düsseldorfer Ausstellung 1902. Weese and Gehler. (Paper read before the Deutsche Beton Verein.) **(51)** Sérial beginning Apr. 14.
Querträger bei Balkenbrücken.* K. Oswald. **(78)** Apr. 23.

Electrical.

Relation of Load Factor to Power Costs. E. W. Lloyd, C. A. S. Howlett and J. M. S. Waring. **(4)** Apr.
Tungsten and other Lamps.* George Loring. **(3)** Apr.
Output and Economy Limits of Dynamo-Electric Machinery.* J. C. Macfarlane and H. Burge. **(77)** Apr.
Transformers: Some Theoretical and Practical Considerations.* A. P. M. Fleming and K. M. Faye-Hansen. **(77)** Apr.

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Electrical—(Continued.)

On a Method of Using Transformers as Choking Coils and its Application to the Testing of Alternators.* J. D. Coales. (77) Apr.
Non-Inductive, Water-Cooled Standard Resistances for Precision Alternating-Current Measurements.* Clifford C. Paterson and E. H. Rayner. (77) Apr.
Commercial Electric Heating. John Roberts. (77) Apr.
The Performance of a Synchronous Commutator.* J. B. Whitehead and L. O. Grondahl. (27) Apr. 15.
The Composition and Durability of Cable Papers. Clayton Beadle and Henry P. Stevens. (73) Serial beginning Apr. 16.
Sioux Falls Hydro-Electric Development.* (27) Apr. 22.
The Effect of the Starting Characteristics upon the Operation of an Induction Motor.* M. Berthold. (27) Apr. 22.
The 45 000-Volt Transmissions of the Compagnie Electrique du Nord, France.* (26) Apr. 23.
Main Switchboard of Pratt Street Power Station, Baltimore.* (17) Apr. 24.
Central Heating and Lighting Plant for the United States Military Academy.* Henry C. Meyer, Jr. (27) Apr. 29; (14) Serial beginning May 1; (64) Apr. 27.
Tests of Watt-Hour Meters. (27) Apr. 29.
Power Costs in a 1 250-Kw. Central Station.* H. S. Knowlton. (13) Apr. 29.
A New System of Wireless Telegraphy used by the Telefunken Company. Count Arco. (73) Apr. 30.
Fundamental Principles in the Design of Spark-Telegraphy Stations. F. Kiebitz. (Abstract from *Elek. Zeit.*) (73) Apr. 30.
Auxiliary Poles for Direct-Current Machines, Their Electrical Design and Method of Testing.* John N. Dodd. (42) May.
Electricity and the Conservation of Energy. Lewis B. Stillwell. (42) May.
Principles of Fault Location; the Use of the Wheatstone Testing Instruments.* Julius Bernstein. (19) May 1.
System of the Lockport Light, Heat & Power Company.* (27) May 6.
The Electrical Equipment of the Gorham Manufacturing Company.* C. E. Fairbanks. (27) May 6.
Electricity in Buffalo, N. Y. George J. Kirchgasser. (27) May 6.
Submerged Wire Rheostat.* P. M. Brown. (27) May 6.
Steam Consumption Tests of a 500-Kw. Turbo-Generator Set.* A. C. Scott. (27) May 6.
Conférence sur la Télégraphie sans Fils.* Ferrié. (43) Jan.
Les Lampes à Incandescence à Filaments Métalliques. Blondel. (From *Bulletin*, Soc. Inter. des Elec.) (33) Apr. 10.
Usine Electrique de la Cie. d'Electricité de Marseille au Cap Pinède.* (33) Apr. 17.
Télégraphie sans Fil par Ondes Dirigeables système Bellini-Tosi.* J. Quinat. (33) Apr. 24.
Elektrisch Angetriebene Kompressoren.* J. Havlicek. (48) Apr. 10.
Das Kraftwerk der A E G-Turbinenfabrik in Berlin.* O. Lasche. (48) Serial beginning Apr. 24.

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The Influence of Midship-Section Shape upon the Resistance of Ships.* D. W. Taylor. (91) Vol. 16.
Further Experiments upon Longitudinal Distribution of Displacement and its Effect upon Resistance.* Herbert C. Sadler. (91) Vol. 16.
Deviation of the Compass Aboard Steel Ships, its Avoidance and Correction.* L. H. Chandler. (91) Vol. 16.
The Influence of Free Water Ballast upon Ships and Floating Docks.* T. G. Roberts. (91) Vol. 16.
Some Recent Inventions as Applied to Modern Steamships.* W. Carlile Wallace. (91) Vol. 16.
Service Test of the Steamship *Harvard* (Parson's Turbine).* C. H. Peabody, W. S. Leland and H. A. Everett. (91) Vol. 16.
Shipbuilding on the Great Lakes.* Robert Curr. (91) Vol. 16.
Centrifugal Pump Fire-Boats.* Charles C. West. (91) Vol. 16.
Sea-Going Suction Dredges.* Thomas M. Cornbrooks. (91) Vol. 16.
The Brazilian Battleship *São Paulo.** (11) Apr. 23.
Marine Steam Turbines. (12) Apr. 30.
The French Dreadnoughts.* (12) Apr. 30.
The Graving Docks of the World.* N. G. Gedye, Assoc. M. Inst. C. E. (12) Serial beginning Apr. 30.
Curtis Turbines for the *North Dakota.** (95) May.
Naval Boilers in Service.* H. C. Binger. (95) May.
Recherches sur la Résistance à la Traction des Bateaux un Nouveau Mesurier de Vitesses.* Barlatier de Mas et Guillet. (43) Jan.
L'Application de la Surchauffe aux Moteurs de Navires.* O.-H. Wildt. (33) Apr. 17.

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Mechanical.

The Theory of Steam Condensers. Thos. C. McBride. (2) Oct., 1908.
Tests of Coal and Briquets as Fuel for House-Heating Boilers. D. T. Randall. (70) Apr.
Coal Briquetting.* Carl Scholz. (4) Apr.
New Silica Brick Plant of the Harbison-Walker Refractories Co.* (76) Apr.
Paul Fuchs' Process of Making Granite Brick.* (76) Apr.
The Conditions Favourable to the Production of Methane.* Thomas Holgate. (66) Serial beginning Apr. 6.
Power System of Louisville Lighting Co.* Osborn Monnett. (64) Apr. 13.
Gas-Producer Central Station at Harvard, Ill.* (27) Apr. 15.
Prony Brakes for Practical Testing.* G. Everett Quick. (19) Apr. 17.
Rolled Manganese Steel Rails; the Manard Rail of the Pennsylvania Steel Company.* (20) Apr. 22.
A Constant-Pressure Internal-Combustion Engine.* H. B. Stilz. (13) Apr. 22.
A Traveling Bridge Suspended from a Cableway for Making Fills.* (13) Apr. 22.
An Electrically Operated Suspended Railway.* Alfred Gradenwitz. (16) Apr. 24.
A New Method in Water Gas Manufacture. Henry I. Lea. (Paper read before the Ill. Gas Assoc.) (24) Apr. 26; (83) Apr. 15.
The Production of Sound Castings. Robert Buchanan. (From *Jour.*, West of Scotland Iron and Steel Inst.) (62) Apr. 26.
Steam Turbines for the Propulsion of Vessels at Moderate Speed.* Yasuzo Wadagaki. (Abstract of paper read before the North-East Coast Inst. of Engrs. and Shipbuilders). (47) Apr. 30.
The Influence of Steam Pressure, Speed of Rotation, and Number of Expansions upon the Mean Effective Pressure and Steam Consumption of Steam Engines.* R. Royds. (Paper read before the Inst. of Engrs. and Shipbuilders in Scotland.) (47) Serial beginning Apr. 30.
Donohoe Coke Co.* C. R. King. (45) May.
The 100-Horse Power Gas Engine, Current British Practice.* T. W. Burt. (10) May.
Principles of Furnace Design for Smokeless Combustion. C. H. McClure. (Paper read before the Amer. Chem. Soc.) (94) May.
Economies in the Manufacture of Iron and Steel.* G. B. Waterhouse. (9) Serial beginning May.
The Renaissance of the Absorption Machine. Jos. H. Hart. (9) May.
Koppers By-Product Coke Oven Plant at Joliet.* R. Gunderson. (83) May 1.
The Traction Elevators in the Singer Building, New York.* (14) May 1.
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The Mills of the Cowell Portland Cement Company.* (14) May 8.
An Investigation of the Effects of Compression and Transmission on the Candle Power and Heat Units of Manufactured Gas.* H. E. Bates and J. R. Rafferty. (Paper read before the Ill. Gas. Assoc.) (24) May 10.
Note sur l'Utilisation Directe du Gaz de Four à Coke dans les Moteurs à Explosion. E. Cuvelette. (32) Feb.
Etude sur le Ressort à Boudin.* M. Astier. (37) Mar. 31.
Remarques sur la Compression des Moteurs à Gaz.* E. Jouguet. (37) Mar. 31.
Les Changements de Vitesse des Automobiles.* E. Girardault et J. Bethenod. (33) Apr. 17.
Gazogène à Combustion Horizontale pour Fours à Cornues d'Usines à Gaz, Système Parsy.* A. Grebel. (33) Apr. 24.
Ueber Turbogebläse. H. Naville. (Paper read before the Eisenhütte Südwest.) (50) Apr. 7.
Neuere Flachregler mit Regelbarer Umlaufzahl.* R. Proell. (48) Apr. 10.
Das Riemengetriebe und die Gehrckenssche Theorie.* Ludwig Benjamin. (48) Apr. 24.

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A New Electrical Hardening Furnace.* E. Sabersky and E. Adler. (Abstract of paper read before the Faraday Soc.) (73) Apr. 16; (11) Apr. 16.
The Mount Morgan Metallurgical Works.* Gerard W. Williams. (16) Apr. 17.
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New Type of Washer for Low-Grade Gold Ores.* Joseph Hyde Pratt. (16) May 8.
Concentrating Difficult Lead Ores at Broken Hill. Gerard W. Williams. (16) May 8.
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Neues über Härteöfen.* D. H. Haedicke. (50) Serial beginning Apr. 28.

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Coal-Dust to Date and its Treatment with Calcium Chloride.* Henry Hall (59) Vol. 31.
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Deep Diamond-Boring at Balfour Mains, Fifeshire. James G. Thomson. (59) Vol. 31.
Sinking Pumps.* Frank S. Mitchell. (Paper real before the Montana Soc. of Engrs.) (1) Mar.
Mining Coal with Machines in England.* George R. Dixon. (16) Apr. 17.
Lowering a Large Pump into a Mine.* George J. Young. (16) Apr. 17.
Development of Dredges for Placer Deposits.* George B. Massey, II. (16) Apr. 24.
Water Losses at Broken Hill. T. H. Palmer. (16) Apr. 24.
Counterbalanced Hoisting.* R. L. Herrick. (45) May.
Three Thousand Horse Power Winding Engine.* J. B. Van Brussel. (16) May 1.
Restoring Dredged Ground.* D. B. Sellars. (Abstract of a report of the Secy. of Mines, Victoria.) (16) May 8.
An Investigation of the Cost of Mining Coal. J. R. Finlay. (16) May 8.
La Station d'Essais du Comité Central des Houillères de France à Liévin (Pas-de-Calais).* (33) Apr. 10.
Elektrisch Angetriebene Kompressoren.* J. Havlicek. (48) Apr. 10.

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Pyrometers for Measurement of High Temperature.* Richard P. Brown. (2) Oct., 1908.
The Patent System in its Relation to Industrial Development. Frederick P. Fish. (42) May.

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The Theory of Economical Road and Street Design. (86) Serial beginning Apr. 14.
The Top Soil Method of Highway Construction. C. M. Strahan (14) Apr. 24.
An Analysis of Highway Traffic in Rhode Island.* Arthur Horace Blanchard, Assoc. M. Am. Soc. C. E. (Paper read before the Amer. Assoc. for the Advancement of Sci.) (13) Apr. 29.
Controlling the Consistency of Bituminous Binders.* C. N. Forrest, Assoc. M. Am. Soc. C. E. (14) May 1.

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Electric Railways in the Ohio Valley between Steubenville, Ohio, and Vanport, Pennsylvania.* George B. Francis, M. Am. Soc. C. E. (54) Vol. 63.
Elevating Tracks of the Philadelphia, Baltimore & Washington Railroad through Wilmington, Del.* H. S. Righter. (2) Oct., 1908.
Car Wheel Forging and Conditions of Steel for High Service.* Jas. H. Baker. (58) Apr.
New Carriages, Great Indian Peninsular Railway.* (21) Apr.
Superheaters (Montcheuil Type) without Headers.* Charles R. King, M. Inst. M. E. (21) Apr.
Tunnel Construction.* W. H. Booth. (21) Apr.
Re-inforced Concrete Sleepers in America.* (21) Apr.
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Electro-Pneumatic Power Signalling at Glasgow Central Station; Caledonian Railway.* (21) Serial beginning Apr.
Developments in Electrification of Railway Terminals. H. H. Evans. (4) Apr.
A Gravel Washing Plant on the Richmond, Fredericksburg and Potomac R. R.* E. M. Hastings. (13) Apr. 15; (18) Apr. 10.
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General Instructions to Locating Engineers Alaska Central Railway. G. A. Kyle. (Abstract of paper read before the Pacific Northwest Soc. of Engrs.) (86) Apr. 21.

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The Problem of Reducing Smoke on Railroads. Alfred W. Gibbs. (From paper for the Amer. Civic Federation.) (13) Apr. 22; (19) Apr. 24.
Report of the Boston Metropolitan Improvements Commission.* (15) Apr. 23.
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Baldwin 213-Ton Articulated Locomotive for Southern Pacific Ry.; the Heaviest Locomotive ever Built.* (13) Apr. 29; (15) Apr. 30; (18) Apr. 24; (25) May.
The Disadvantages of Concrete Foundations for Railway Crossings.* R. P. Black. (13) Apr. 29.
Heavy Passenger Locomotives of the 4:6:2 Class; Western Railway of France.* (13) Apr. 29.
The Chamonix-Martigny Electric Railway.* C. Van Langendonck. (15) Apr. 30.
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Side-Rod Single-Phase Locomotive.* (27) May 6; (14) May 8; (17) May 8.
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Traffic Studies.* Ray Morris. (15) Serial beginning May 7.
Train Resistance.* A. Stucki. (15) May 7.
Notice sur la Construction des Chemins de Fer Départementaux de Cher.* Mayer. (43) Jan.
Chemins de Fer d'Interêt Local et Tramways; Note sur une Anomalie qui Peut Résulter de Plusieurs Conventions Récemment Adoptées. H. Heude. (43) Jan.
Les Procédés les Plus Récents d'Enclenchements des Signaux et des Aiguilles.* A. Moutier. (32) Feb.
Note sur les Conditions de Construction des Locomotives aux Etats-Unis, en Allemagne et en France. Marcel Bloch. (38) Apr.
Sablière pour Locomotives et Voitures Automotrices, Système Lambert.* (34) Apr.
Umgestaltung der Bahnanlagen in und bei Leipzig.* Bischof and Boltze. (49) Serial beginning Vol. 4-6, 1909.
Der Bahnhof Vohwinkel und Seine Entwicklung.* Claus. (49) Vol. 4-6, 1909.
Entwerfen und Bau von Lokomotivschuppen.* Cornelius. (49) Vol. 4-6, 1909.
Das Wanken der Lokomotiven unter Berücksichtigung des Federspieles. J. Jahn. (48) Serial beginning Apr. 3.
Die Elektrischen F4/4-Lokomotiven am Simplon.* E. Thomann und K. Schnetzler. (48) Serial beginning Apr. 17.
Die Neuen 2/5-Gekuppelten Schnellzuglokomotiven der Preussischen Staatsbahn.* Metzeltin. Serial beginning Apr. 24.

Railroad, Street.

Electric Interlocking and Signaling System for the Street Railways at the Union Station Plaza, Washington, D. C.* (13) Apr. 15.
The 600-Volt Lighting System with Grounded Circuits used by the Brooklyn Rapid Transit Company. (17) Apr. 17.
Important Automatic Switching Installations in Washington, D. C.* (17) Apr. 17.
Ventilation of the Washington Street Tunnel, Boston.* (14) Apr. 24.
T-Rail in Paved Streets at Charlotte, N. C.* (17) Apr. 24; (14) Apr. 24.
Heating a Trolley-Car Barn.* J. Irving Brewer. (13) Apr. 29.
Large New Car House of Chicago City Railway.* (17) May 8.
Results of a Study of Car Ventilation in Chicago. (17) May 8.
Le Chemin de Fer Métropolitain de Paris.* R. Godfernaux. (38) Apr.

Sanitary.

Sewerage and Drainage Improvements at Cairo, Illinois.* (14) Apr. 17.
Specifications and Bids for a Refuse Incinerating and Power Generating Plant for Milwaukee, Wis. (13) Apr. 22.

*Illustrated.

Sanitary—(Continued).

Sewage Purification Works at Mt. Vernon, N. Y.* Geo. L. Robinson, Assoc. M. Am. Soc. C. E. (13) Apr. 29.
Methods of Sewer Construction in Clinton, Iowa.* Charles P. Chase. (Abstract of paper read before the Iowa Soc. of Engrs.) (60) May.
Legal Status of Works Producing Noxious Gases. Charles Baskerville. (16) May 1.
The Louisville Sewerage Improvements.* Roger DeL. French. (14) May 1.
Fundamental Principles of Sewage Purification on Land. Rudolph Hering, M. Am. Soc. C. E. (13) May 6.
Einige Mitteilungen über die Kanalisation von Belgrad und den Bau des Hauptsammlers der Oberen Zone.* Ludwig Roth. (78) Apr. 2.
Die Heizungsanlage im Fabrikgebäude der Deutschen Gasglühlicht-Aktiengesellschaft (Auergesellschaft) in Berlin. Karl Brabbée. (48) Apr. 3.
Die Trennung von Staub und Luft in Absauganlagen. Ewald Mees. (48) Apr. 17.

Structural.

Foundations for the New Singer Building, New York City.* T. Kennard Thomson, M. Am. Soc. C. E. (54) Vol. 63.
Waterproofing, an Engineering Problem. Myron H. Lewis. (2) Oct., 1908.
Manganese Steel.* W. S. Potter. (4) Apr.
The Vaughan System of Reinforced Concrete Construction.* (67) Apr.
Methods and Cost of Constructing Frame Cottages and Service Buildings for the Camaguey, Cuba, Waterworks.* Henry A. Young. (86) Apr. 14.
Some Methods in Competitive Reinforced Concrete Design. Ernest McCullough. (Abstract of paper read before the Armour Inst. of Tech.) (14) Apr. 17; (86) Apr. 14.
The Reconstruction in Reinforced Concrete of the Large Manufacturing Plant of J. B. King & Co., Staten Island, N. Y.* Arthur G. Hoadley. (13) Apr. 22.
Guest's Law of Combined Stress.* C. A. Smith. (11) Apr. 23.
The Building Exhibition. (12) Serial beginning Apr. 23.
Corrosion of Iron and Steel. Edward Crowe. (Abstract of paper read before the Cleveland Inst. of Engrs.) (12) Apr. 23.
Tables for Calculating Inertia of Built-up Girders.* H. F. Harris. (47) Apr. 23.
The Knickerbocker Trust Building Substructure.* (14) Apr. 24.
The Cummins Station Warehouse, Nashville, Tennessee.* (13) Apr. 24.
Some Experiments on Impact.* J. E. Sears. (11) Serial beginning Apr. 30.
The Dangers of Breeze Concrete.* (11) Apr. 30.
A Large Concrete and Timber Coal Pocket.* (14) May 8.
Foundations of the Northwestern Railway Terminal, Chicago.* (14) May 8.
Influence du Traitement Thermique sur les Déformations Linéaires des Aciers. Demozay. (93) Apr.
Ergänzungen zu der Berechnung der Betoneisen-Konstruktionen. Heinrich Pilgrim. (81) Vol. 3, 1909.
Bestimmung der Gesamtbasizität des Zementes. Eduard Toth. (80) Serial beginning Apr. 10.
Das Neue Theater in Kopenhagen.* E. Suenson. (78) Apr. 23.
Erdbebensichere Bauten.* F. von Emperger. (78) Apr. 23.

Water Supply.

The Status of the Wave Motor.* James T. Barkelew. (64) Apr. 13.
Irrigation Works in the Hawaiian Islands.* M. M. O'Shaughnessy, M. Am. Soc. C. E. (13) Apr. 15.
The Early Construction Methods at the Plant of the Central Georgia Power Company.* (14) Apr. 17.
Sedimentation of the Water-Supply and its Relation to Typhoid Fever in Washington, D. C.* Francis F. Longley. (13) Apr. 22.
The Valuation for City Purchase of the Property of the Waterloo (Ia.) Water-Works Co. A. Marston, M. Am. Soc. C. E. (13) Apr. 22.
The Garden City Pumping Plant.* F. W. Hanna. (14) Apr. 24.
Swedish Hydro-Electric Power-Plants.* John Geo. Leigh. (10) May.
Impurities Causing Scale and Corrosion. J. C. William Greth. (Abstract of paper read before the Amer. Inst. Chem. Engrs.) (94) May.
The Design of Steel Pipes for Hydro-Electric Plants.* Richard Muller. (14) May 1.
The High Head Hydro-Electric Development at Loch Leven, Scotland.* J. Murray S. Culbertson. (14) May 1.
Recherches et Observations Récentes sur le Belier Hydraulique.* Goupil. (43) Jan.
Les Nouveaux Filtres à Sable non Submergé système de Mm. le Docteur P. Miquel et Mouchet.* Félix Marboutin. (32) Feb.
Calcul des Tuyaux en Béton Armé soumis à des Efforts de Compression et de Flexion.* P. Caufourier. (33) Apr. 3.
Perturbations Produites par la Fermeture des Robinets-Vannes sur le Fonctionnement d'une Distribution d'Eau.* Henry Dide. (33) Apr. 24.
Die Wasserversorgungs-Anlage für Apulien. Dr. Brandau. (Paper read before the Arch.-u. Ing.-Verein zu Cassel.) (51) Serial beginning Apr. 10.

*Illustrated.

Waterway.

The Low Stages of Lakes Huron and Michigan.* C. E. Grunsky, M. Am. Soc. C. E. (54) Vol. 63.
The Floods of the Mississippi Delta: Their Causes and Suggestions as to their Control. William D. Pickett, M. Am. Soc. C. E. (54) Vol. 63.
The Improvement of the Ohio River.* William L. Sibert, M. Am. Soc. C. E. (54) Vol. 63.
The Colorado River Breach and Closure.* (12) Apr. 9.
The Proposed National Ship Canal in Scotland.* (11) Apr. 23.
Construction of a Sea Wall at Fort Morgan, Alabama.* (14) Apr. 24.
Extraordinary Ice Jams at Niagara Falls.* (13) Apr. 29.
The Present Condition of the Suez Canal.* (12) Apr. 30.
A Dredging Equipment in the Himalaya Mountains.* (95) May.
An Eight-Yard Dipper Dredge for the Cuban Government.* (95) May.
Design of Hulls for Hydraulic Cutter Dredges. E. N. Percy. (95) May.
The Bates Electrically Driven Hydraulic Dredger.* (95) May.
Lock No. 3, Cumberland River.* William W. Harts, M. Am. Soc. C. E. (13) May 6.
The Design of Cable Stations for River Measurements.* J. C. Stevens. (13) May 6.
Built-Up Concrete Slab Piles and Pile Protection.* (14) May 8.
New Municipal Piers in Baltimore Harbor.* (14) May 8.
Les Echelles à Poissons et leur Application aux Barrages de Meuse et d'Ourthe.* G. Denil. (30) Apr.
Errichtung eines Kais mittels Eisenbeton-Schwimmblöcken in Port Said.* Walther Stross. (78) Apr. 2.
Neuere Wehrbauten aus Eisenbeton.* F. W. Otto Schulze. (78) Serial beginning Apr. 2.
Vom Panamakanal.* Max Frhrn. von Wendland. (51) Serial beginning Apr. 7.

*Illustrated.

PROCEEDINGS

OF THE

AMERICAN SOCIETY

OF

CIVIL ENGINEERS

(INSTITUTED 1852)

VOL. XXXV—No. 6.

AUGUST, 1909.

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NEW YORK 1909

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ON ENGINEERING EDUCATION: Desmond FitzGerald, Benjamin M. Harrod, Onward Bates, D. W. Mead, Charles Hansel.

ON STEEL COLUMNS AND STRUTS: Austin L. Bowman, Alfred P. Boller, Emil Gerber, Charles F. Loweth, Ralph Modjeski, Frank C. Osborn, George H. Pegram, Lewis D. Rights, George F. Swain, Emil Swensson, Joseph R. Worcester.

ON BITUMINOUS MATERIALS FOR ROAD CONSTRUCTION: W. W. Crosby, A. W. Dean, H. K. Bishop, A. H. Blanchard.

The House of the Society is open from 9 A.M. to 10 P.M. every day, except Sundays, Fourth of July, Thanksgiving Day and Christmas Day.

HOUSE OF THE SOCIETY—220 WEST FIFTY-SEVENTH STREET, NEW YORK.

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AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PROCEEDINGS

This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

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MINUTES OF MEETINGS OF THE SOCIETY

May 19th, 1909.—The meeting was called to order at 8.30 P. M.; Vice-President George H. Pegram in the chair; Chas. Warren Hunt, Secretary; and present, also, 137 members and 24 guests.

A paper by C. E. Grunsky, M. Am. Soc. C. E., entitled "The Sewer System of San Francisco, and a Solution of the Storm-Water Flow Problem," was presented by the author and illustrated with lantern slides. The Secretary presented written communications on the subject by Messrs. Robert G. Dieck and W. C. Hammatt, and the paper was discussed orally by Messrs. C. E. Gregory, E. Kuichling, and Kenneth Allen.

A paper by W. S. McFetridge, M. Am. Soc. C. E., entitled "Some Extensive Railroad Surveys, and Their Cost per Mile," was presented by the Secretary, and discussed by F. Lavis, M. Am. Soc. C. E., who illustrated his remarks with lantern slides. A written communication on the subject, from Clinton S. Bissell, M. Am. Soc. C. E., was read by the Secretary, and E. W. Lewis, M. Am. Soc. C. E., discussed the subject orally.

The Secretary announced the death of WALTER LORTON DUSENBERRY, elected Junior November 5th, 1890; died May 13th, 1909.

Adjourned.

June 2d, 1909.—The meeting was called to order at 8.30 P. M.; Vice-President George H. Pegram in the chair; Chas. Warren Hunt, Secretary; and present, also, 136 members and 38 guests.

A paper, entitled "Tests of Built-Up Steel and Wrought-Iron Compression Pieces," by Arthur N. Talbot, M. Am. Soc. C. E., and Herbert F. Moore, Esq., was presented by the Secretary, who also read a communication on the subject from John C. Moses, M. Am. Soc. C. E.

A paper by Henry Japp, M. Am. Soc. C. E., entitled "Caisson Disease and Its Prevention," was presented by the author and illustrated with lantern slides. Written communications on the paper from Dr. J. S. Haldane, and Dr. Fred. L. Keays, were read by the Secretary, and the subject was discussed orally by Messrs. William L. Saunders, Albert J. Loomis, Walton I. Aims, and Seward Erdman.

The Secretary announced the election of the following candidates by the Board of Direction on June 1st, 1909:

AS MEMBERS.

WILLIAM COULSON ARMSTRONG, Chicago, Ill.
BERTRAM BREWER, Waltham, Mass.
ISAAC GLIDDEN CALDERWOOD, Brewerton, N. Y.
AUSTIN BRADSTREET FLETCHER, Boston, Mass.
GEORGE EDWARD HARKNESS, Boston, Mass.
WILLIAM SHERMAN HEWETT, Minneapolis, Minn.
HOWARD MURFREE JONES, Nashville, Tenn.
JOHN BUCK LEONARD, San Francisco, Cal.
WYNN MEREDITH, San Francisco, Cal.
WILLIAM SELBY MITCHELL, St. Louis, Mo.
RICHARD MONROE, Rock Island, Ill.
FREDERICK YANCY PARKER, St. Louis, Mo.
JOHN HOWE PEYTON, Louisville, Ky.
WILLIAM RENTON PHILLIPS, Portland, Ore.
EARLE WILLOUGHBY SAYLES, Watertown, N. Y.
CHRISTOFFER SCHROEDER, Chicago, Ill.

David Gorton Thomas, Denver, Colo.
Julius Edgar Willoughby, Louisville, Ky.

As Associate Members.

Harry Wray Claybaugh, Franklin, Pa.
Bernard Alfred Etcheverry, Berkeley, Cal.
Harley Edgar Frye, Zanesville, Ohio.
Edward Charles Gersbach, Helena, Mont.
Lewis Merritt Gram, Toledo, Ohio.
Harry LeGrand Hilton, Cornwall-on-Hudson, N. Y.
Charles Eugene Johnston, Kansas City, Mo.
Olaf Laurgaard, Conconully, Wash.
William Meier, Chicago, Ill.
John Henry Myers, Harrisburg, Pa.
Samuel Donald Newton, Knoxville, Tenn.
Howard Charles Paddock, Brooklyn, N. Y.
Alan Bigelow Perley, Mt. Airy, Pa.
John Hudson Poole, Washington, D. C.
Henry Blanchard Pratt, Waltham, Mass.
Theodore Eckford Rhoades, New York City.
Edward Henry Rockwell, Tufts College, Mass.
Garret Edward Tilt, Paterson, N. J.
Warren Chamberlain Tudbury, Sanborn, N. Y.
Norman Rand Willard, Douglas, Wyo.
Adolph Yappen, Chicago, Ill.

As Juniors.

Herman Chafetz, Buffalo, N. Y.
Clesson Herbert Field, South Bethlehem, Pa.
Leo Daniel Graham, St. Ignatius, Mont.
Howard Kingsbury Holland, Ann Arbor, Mich.
William James Horrigan, Wilmington, Del.
Frank Ray Howe, Flushing, N. Y.
Franklin Johnson Howes, Rochester, N. Y.
William Richard Hughes, Jr., Monaca, Pa.
Albert August Krieger, Louisville, Ky.
Robert Sherman Larimer, Estes Park, Colo.
Karl Wolfgang Lemcke, Ambridge, Pa.
George Higgins Myers, Denver, Colo.
John Robert Nichols, Brookline, Mass.
Alfred Worcester Nordwell, Cambridge, Mass.
Gilbert Small, Waltham, Mass.
Jonathan Ernest Teal, Lafayette, Ind.
Clement Clarence Williams, Boulder, Colo.
Thomas Judson Wright, Jr., Urbana, Ill.

The Secretary announced the transfer of the following candidates by the Board of Direction on June 1st, 1909:

FROM ASSOCIATE MEMBER TO MEMBER.

JACOB LOUIS BAUER, Elizabeth, N. J.
CHARLES ROBERT BETTES, Far Rockaway, N. Y.
CHARLES CORWIN CREW, Bay City, Mich.
HARRY HARDY, Colon, Panama.
JOHN CLAYTON HOYT, Washington, D. C.
CHARLES GILMAN HYDE, Berkeley, Cal.
CHARLES GILLINGHAM MOORE, Buffalo, N. Y.
CHAUNCY RUSCH PERRY, Waltham, Mass.

FROM JUNIOR TO MEMBER.

JOHN NELSON HAYWARD CORNELL, JR., New York City.

FROM JUNIOR TO ASSOCIATE MEMBER.

CHARLES WHITNEY TILLINGHAST BARKER, JR., Philadelphia, Pa.
HERBERT WATSON HATTON, Wilmington, Del.
GEORGE SAMUEL HILL, San Francisco, Cal.
MATTHEW DETOBIN KELLEY, Lander, Wyo.

Adjourned.

ELECTIONS BY THE BOARD OF DIRECTION, JULY 1st, 1909

At the Business Meeting at the Annual Convention, July 7th, 1909, the Secretary announced the following elections, transfers, and deaths:

AS MEMBERS.

HORACE THOMAS CAMPION, Philadelphia, Pa.
ROBERT ROGERS EVANS, Haverhill, Mass.
WILLIAM HENRY FENN, Wilmington, Del.
JOHN BERTON FERGUSON, Lima, Ohio.
HENRY FOX, Baltimore, Md.
JAMES ADAMS FRENCH, Engle, N. Mex.
ARTHUR GRACE, Medicine Hat, Alta., Canada.
ERWIN ERNEST HASLAM, Niagara Falls, N. Y.
JOHN BENJAMIN HITTELL, Chicago, Ill.
WILLIAM ATLEE JAMES, Kenora, Ont., Canada.
EDWARD NELSON LAKE, Wheaton, Ill.
CHARLES THOMAS MAIN, Boston, Mass.
LAURENCE BRADFORD MANLEY, West Roxbury, Mass.
EDGAR DARWIN MARTIN, Chicago, Ill.
ARTHUR O'BRIEN, Utica, N. Y.
LOGAN WALLER PAGE, Washington, D. C.

George Hamilton Rogers, Tuckahoe, N. Y.
Grant Rohrer, New York City.
Louis Keegan Rourke, Empire, Canal Zone, Panama.
Hugh McGehee Taylor, Mexico City, Mexico.
William Martin Torrance, New York City.
Rudolph Warner Van Norden, San Francisco, Cal.
Friend Pitts Williams, Amsterdam, N. Y.
Philip Brundage Windsor, New York City.

As Associate Members.

William Nicholas Beach, New York City.
Louis Andrew Burns, Rome, N. Y.
Duncan Hugh Campbell, Butte, Mont.
Edward Gilman Carey, New York City.
Clermont C. Covert, Albany, N. Y.
John Charles Davis, Milwaukee, Wis.
Vahram Yettvart Davoud, Provo, Utah.
Thomas Jefferson Edelen, Winnipeg, Man., Canada.
Milton Harvey Freeman, New York City.
Frederick Louis Gagnon, New York City.
Henry Lilburn Gray, Olympia, Wash.
Charles William Hardt, Camp Hill, Pa.
Nicholas Hunter Heck, Key West, Fla.
Clyde Leslie Huff, Salt Lake City, Utah.
Hinman Barrett Hurlbut, New Haven, Conn.
Edwin Gustav Kastenhuber, Jr., Bordentown, N. J.
Howard B. Merrick, Ann Arbor, Mich.
Clair Brandon Patterson, Darrowville, Ohio.
Joel Marsh Pratt, Mobile, Ala.
Ray Howard Rice, Norfolk, Va.
Felix Perceval St. Hill, Hamilton, New Zealand.
Stuart Sims, Moscow, Idaho.
Edward Augustus Southworth, Honolulu, Hawaii.
Harry Clinton Sweeney, Brooklyn, N. Y.
William Edward Thomas, Jamaica, N. Y.
Paul Alphonse Gaillard Tilmont, Nimrod, Mont.
Albert Edwin Waldron, Dallas, Tex.
Jacob Paul Jones Williams, Swissvale, Pa.
Leo P. Winterhalter, Cleveland, Ohio.

As Juniors.

James Everett Beswick, Cold Spring, N. Y.
Arnold Goodwin Chapman, Albany, N. Y.
Pinkney Edward Cunningham, Vicksburg, Miss.
Frank Haskell Densler, Poughkeepsie, N. Y.

Earl Edwin Erdmann, Beverly, Wash.
Harry Carter Gardner, Boulder, Colo.
Gordon Burnett Gifford Hull, Monterey, Mexico.
Stanley Wallace Moore, Manila, Philippine Islands.
Robert Isham Randolph, Chicago, Ill.
Shaler Gordon Smith, Hoboken, N. J.
Adolph Teichert, Jr., Sacramento, Cal.
Ferd. Clark Wintermute, Wilkes-Barre, Pa.

Transfers from Associate Member to Member.

James Simpson Browne, Providence, R. I.
George Heckman Burgess, Albany, N. Y.
Harry Gilbert Burrowes, New York City.
Alvah Benjamin Diehr, Memphis, Tenn.
George Harrison Fenkell, Detroit, Mich.
George Estyn Goodwin, Family, Mont.
Arthur Newhall Johnson, Springfield, Ill.
Francis Mason, New York City.
Hugh Percival Raikes, Birmingham, England.
Francis Libby Sellew, Yuma, Ariz.
Morton Loudon Tower, San Francisco, Cal.

Transfers from Junior to Associate Member.

Elbert Allan Gibbs, Wilkinsburg, Pa.
Walter Joseph Gillen, Brown Station, N. Y.
Thomas Benton Hunter, Monterey, Cal.
Clifford Marshall King, Sandusky, Ohio.
Isaac Solon Matlaw, Schenectady, N. Y.
George Edward Pellissier, New York City.
Carl Howell Reeves, Seattle, Wash.
Horace West Sheley, Salt Lake City, Utah.
Roy Adolf Swartwout, Omaha, Nebr.

Deaths.

Henry Furlong Baldwin, elected Associate Member December 7th, 1892; Member February 6th, 1895; died June 17th, 1909.

Frank Leslie Davis, elected Member September 6th, 1905; died June 9th, 1909.

John Bagley Dimmick, elected Member July 10th, 1907; date of death unknown.

George Samuel Gatchell, elected Member May 7th, 1884; died June 22d, 1909.

William Charles Kernot, elected Member March 6th, 1889; died March 14th, 1909.

FORTY-FIRST ANNUAL CONVENTION
HELD AT BRETTON WOODS, N. H., JULY 6th–9th, 1909.

FIRST SESSION*

Tuesday, July 6th, 1909.—The meeting was called to order at 10 A. M.; John W. Ellis, M. Am. Soc. C. E., Chairman of the Local Committee of Arrangements, in the chair; Chas. Warren Hunt, Secretary; and present, also, about 300 members and guests.

Mr. Ellis introduced his Excellency, Governor Henry B. Quinby, who welcomed the Society to the State of New Hampshire. Onward Bates, President, Am. Soc. C. E., thanked the Governor for his cordial welcome, and then delivered the Annual Address.†

BUSINESS MEETING

Second Session of the Convention, Wednesday, July 7th, 1909.—The meeting was called to order at 8 P. M.; President Onward Bates in the chair; Chas. Warren Hunt, Secretary; and present, also, about 150 members.

The Secretary presented a report of the suggestions received from members as to the place and time for holding the Annual Convention of 1910.‡

On motion, duly seconded, the matter of the selection of the place and time for holding the next Annual Convention was referred to the Board of Direction with power.

The Secretary presented a report of the Board of Direction on the method of appointing the Nominating Committee.§

In reference to the Special Committee on the Status of the Metric System, the Secretary explained that when the committee had presented its report to the Annual Meeting, January 20th, 1909, it had not asked to be discharged, but, through an error, it was discharged. The matter having been brought to the attention of the Board of Direction subsequently, the Board had reappointed the committee.

In the matter of the Progress Report of the Special Committee on Concrete and Reinforced Concrete, the Secretary read discussions by Messrs. John S. Sewell, Clarence W. Noble, and S. Bent Russell,‖ and Messrs. J. F. O'Rourke and J. R. Worcester‖ discussed the report orally.

On motion, duly seconded, it was ordered that the report, and all discussion thereon, together with any reply to the discussion made by the committee, should be received and printed in *Transactions*.

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* For the report in full of this and the Business Meetings, see page 276.

† *Transactions*, Am. Soc. C. E., Vol. LXIV, p. 567.

‡ See page 278.

§ See page 282.

‖ These discussions will be printed in the September, 1909, number of *Proceedings*.

The Secretary announced the elections and transfers by the Board of Direction on July 1st, 1909.*

The Secretary announced the death of several members.†

On motion, duly seconded, it was ordered that the programme of the Convention be changed, and that the discussion of professional topics be taken up at 2 P. M., July 8th, and that the Reception and Ball take place in the evening of that day.

Adjourned.

Third Session, Thursday, July 8th, 1909.—The meeting was called to order at 2.45 P. M.; President Onward Bates in the chair; Chas. Warren Hunt, Secretary; and present, also, about 60 members.

Discussion on the topic, "Fire Resistant Construction of Buildings," was opened by a communication from Herbert M. Wilson, M. Am. Soc. C. E., presented by Richard L. Humphrey, M. Am. Soc. C. E. The topic was discussed orally by Messrs. R. W. Lesley, R. L. Humphrey, and S. Whinery.

Discussion on the topic, "Impurities in Sand for Concrete," was opened by Sanford E. Thompson, M. Am. Soc. C. E., and was discussed further by Messrs. R. W. Lesley, C. P. Howard, R. L. Humphrey, G. S. Davison, S. Whinery, C. M. Mills, T. F. Richardson, and Sanford E. Thompson.

The third topic for discussion, "Has Experience Demonstrated that the Oiling of Roads is the Most Satisfactory or Economical Method of Preventing Dust and Preserving the Road Surface?" was presented by S. Whinery, M. Am. Soc. C. E., and was discussed further by G. S. Davison, M. Am. Soc. C. E.

Adjourned.

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* See pages 270–272.

† See page 272.

OF THE BOARD OF DIRECTION

(Abstract)

June 1st, 1909.—Vice-President Pegram in the chair; Chas. Warren Hunt, Secretary; and present, also, Messrs. Harrison, Kittredge, Knap, Macdonald, Noble, Stuart, Swensson, Thompson, Tillson, and Wilkins.

Ballots for Membership were canvassed, resulting in the election of 18 Members, 21 Associate Members, and 18 Juniors, the transfer of 8 Associate Members to the grade of Member, 1 Junior to the grade of Member, and 4 Juniors to the grade of Associate Member.

A report of the Board to the Society on the method of the appointment of the Nominating Committee was considered.

Director George W. Kittredge was appointed to represent the Society at the Inauguration of Richard Cockburn Maclaurin, as President of the Massachusetts Institute of Technology, to be held June 7th, 1909.

Applications were considered, and other routine business was transacted.

Adjourned.

July 1st, 1909.—A Special Meeting of the Board to canvass ballots for membership, to consider applications for membership, and to consider the matter of the method by which the Nominating Committee is appointed, was held at 8.20 P. M., July 1st, 1909, President Bates in the chair; Chas. Warren Hunt, Secretary; and present, also, Messrs. Hazen, Pegram, Stuart, Tillson, and Williams.

Ballots for Membership were canvassed, resulting in the election of 24 Members, 29 Associate Members, and 12 Juniors, and the transfer of 11 Associate Members to the grade of Member, and 9 Juniors to the grade of Associate Member.

A report of the Board in the matter referred to it by the Annual Meeting, concerning the method of the appointment of the Nominating Committee, was adopted.*

Applications were considered.

Adjourned.

July 6th, 1909.—The Board met, as required by the Constitution, at the Annual Convention at Bretton Woods, N. H., at 8.50 P. M., July 6th, 1909, President Bates in the chair; Chas. Warren Hunt, Secretary; and present, also, Messrs. Brackett, Churchill, Hodgdon, Horton, Pegram, Stearns, Stuart, Swain, Swensson, Talbot, Thompson, and Williams.

Matters of general interest were discussed.

Adjourned.

* See page 282.

REPORT IN FULL OF THE FIRST SESSION OF THE FORTY-FIRST ANNUAL CONVENTION, AND OF THE BUSINESS MEETING, HELD AT THE MOUNT WASHINGTON HOTEL, BRETTON WOODS, N. H.

FIRST SESSION

Tuesday, July 6th, 1909.—The meeting was called to order at 10 A. M.; John W. Ellis, M. Am. Soc. C. E., in the chair; Chas. Warren Hunt, Secretary; and present, also, about 300 members and guests.

JOHN W. ELLIS, M. AM. SOC. C. E.—As Chairman of the Local Committee of Arrangements, the duty of calling this convention to order has been assigned to me, and I do now declare that the Annual Convention of the American Society of Civil Engineers is now assembled and ready for business.

The principal function of a local committee is to see that every member and his family enjoy themselves to the utmost extent during the days of the Convention. In order to have your hearty co-operation, the committee would be pleased to have the Secretary, Mr. Hunt, make a few announcements.

THE SECRETARY.—Mr. Chairman, in the printed programme for to-day there is scheduled an excursion through Crawford Notch. The members of the local committee are not only expert in making you happy, but are also weather-wise, and they have decided that this would be a very good day to go to the top of Mount Washington. It has been decided, therefore, to change the programme in that respect, and to let those who wish go to the top of Mount Washington to-day instead of later. The train will accommodate about two hundred, and if there are more than that, the remainder, as well as those who have been up the mountain and do not want to go again, can go on the Crawford Notch trip.

The train for the summit leaves the Boston and Maine station at two o'clock. Tickets should be secured at the hotel office here, and are $3 each for the round trip. The Crawford Notch trip requires that you leave the hotel at 12.45 P. M., and the purchase of a ticket at the hotel, which is 50 cents, and then pay your carriage fare, which is $1, to the carriage agent. The trip to the Crawford Notch is described in the printed programme.

The original programme stated that the Boston Society of Civil Engineers has kindly undertaken to make arrangements for an automobile trip about Boston, for members who may wish to visit that city after the close of the Convention, and, later, they will be glad to furnish facilities to our members to visit any engineering works in the vicinity.

Now, the committee is very desirous of knowing how many wish to take advantage of this offer, and therefore it requests that all such kindly register at the Secretary's office, or give their names to Mr. Dexter Brackett, who is here.

MR. ELLIS.—Mr. President and gentlemen: We have met this year in Annual Convention in a beautiful spot, surrounded by the grand old mountains which are only to be had in the State of New Hampshire, and the American Society of Civil Engineers is to be congratulated that we have with us to-day the Chief Executive of that State. It is with the greatest pleasure, and I consider it a special privilege, that I am able to introduce to you one who some forty-six years ago was associated with me in the acquisition of knowledge in an institution of this State.

He has acquired knowledge; he has had preference accorded to him, and it is my privilege to introduce to you at this time Governor Henry B. Quinby of New Hampshire.

Address of Welcome.

GOVERNOR QUINBY.—Mr. Chairman and gentlemen: As Chief Executive of New Hampshire, it gives me pleasure to extend a cordial welcome to our State to the American Society of Civil Engineers. I am especially glad to meet you because my father was a member of your profession, and, it is said, an able one.

Our State Engineer is also a member of your Society, and he has made good as an engineer in charge of our State roads, and the last, but not the least, of those three reasons, is that I consider your profession the chief factor in the building up of this great country of ours, and in widening the fame of our nation.

The American engineer recognizes no difficulties too great to be overcome. He not only initiates but he carries through to successful termination; and the achievements of the American engineer are known to the civilized world. I am glad to welcome you here to this particular spot of our beautiful State, surrounded as it is by the magnificent hills in which we take so much pride.

At the entrance to two of these defiles have been placed stone sentinels by the Supreme Engineer of the Universe, and I feel that when your eyes rest especially upon that one guardian at Profile Notch, there will come into your mind the thought that that face has its gaze fixed upon eternity as it stands upon that lofty height, silent and inscrutable; and you will consider, as you gaze, the wonderful events that have occurred since he was placed on guard, ages and ages ago.

But, ladies and gentlemen, I feel it honor enough to meet this distinguished body. I do not feel that I have the right to take up the time which you might better devote to seeing the beauties of these regions. I believe that as you go from here your visit to the White Hills will be forever a pleasant memory, and I certainly wish you, ladies and gentlemen, a most profitable and a most happy occasion.

ONWARD BATES, PRESIDENT, AM. SOC. C. E.—Your Excellency: In the name of the American Society of Civil Engineers, I desire to thank you for your cordial greeting and the tribute which you have paid to our profession. We are now prepared to enjoy ourselves. We have concluded that our Society has made a wise choice in selecting this place for our Convention, and your gracious words have confirmed that conclusion. We are now going to feast our eyes on this beautiful scenery, to breathe this delightful air, to drink your pure mountain water, and to enjoy to the full this wonderful hotel, with all its comforts; and, for the present at least, we shall forget that we will ever have to leave here, but when that inevitable time does come, we shall take away with us pleasant and enduring memories of Bretton Woods.

The President then delivered the Annual Address.*

BUSINESS MEETING

Business Meeting called to order.

Wednesday, July 7th, 1909.—President Onward Bates in the chair; Chas. Warren Hunt, Secretary; and present, also, about 150 members.

THE PRESIDENT.—The meeting will come to order. The first item of business this evening is to consider the time and place for holding the next Annual Convention. I will ask the Secretary to read the suggestions which have been received from members.

Time and Place for Convention of 1910.

THE SECRETARY.—The following is a report of the suggestions received from members of the Society as to the time and place for holding the next Annual Convention:

"As to the place for holding the next Annual Convention:

"Total number of suggestions received, 268, as follows:

Duluth, Minn.	21	Boston, Mass.	7
San Francisco, Cal.	21	Niagara Falls, N. Y.	7
Seattle, Wash.	19	New York City	7
New Orleans, La.	13	Toledo, Ohio	6
Chicago, Ill.	12	Portland, Ore.	6
Washington, D. C.	12	Pittsburg, Pa.	5
Isthmus of Panama	11	Buffalo, N. Y.	5
Detroit, Mich.	10	Mackinac Island, Mich.	4
Los Angeles, Cal.	10	Quebec, Canada	4
Atlantic City, N. J.	10		

"The following places have received three votes each:

Baltimore, Md.	Philadelphia, Pa.
Cape May, N. J.	St. Paul, Minn.
St. Louis, Mo.	Milwaukee, Wis.
Salt Lake City, Utah.	

* *Transactions*, Am. Soc. C. E., Vol. LXIV, p. 567.

"The following places have received two votes each:

Atlanta, Ga.
Denver, Colo.
Havana, Cuba.
Virginia Hot Springs.
Montreal, Canada.
Old Point Comfort, Va.
Rochester, N. Y.
Saratoga Springs, N. Y.
Sault Ste. Marie, Mich.
Indianapolis, Ind.
Minneapolis, Minn.
Ottawa, Canada.

"The following places have received one vote each:

Albany, N. Y.
Asheville, N. C., or some suitable mountain resort in North Carolina.
Bar Harbor, Me.
Boise, Idaho.
Catskill Mts.
Chattanooga, Tenn.
Colorado Springs, Colo.
Delaware Water Gap, Pa.
Fort Smith, Ark.
Galveston, Tex.
Great Falls, Mont.
Honolulu, Hawaii.
Kansas City, Mo.
Lake Mohonk, N. Y.
Mackinaw, Mich.
Madison, Wis.
Mammoth Cave, Ky.
Blue Mt. House, Md.
Narragansett Pier, R. I.
San Juan, Porto Rico.
Spokane, Wash.
Vancouver, Canada.
Yosemite Valley, Cal.
Watch Hill, R. I.
Yellowstone National Park.
Eastern New York State.
Some mountain hotel between Asheville and Lake Erie.
Any place west of Pittsburg that has adequate accommodations.
One of the finished and operating reclamation projects.
Some point in Western Europe.
Grand Cañon of Arizona.
Charter Lake Steamer to visit Cleveland, Detroit, Soo., etc.
Some point on the Great Lakes.

"Time for holding the Convention:

January	1
February	2
March	3
April	7
May (all parts of May are represented in this count)	18
June	71

as follows:

Early June	4
Middle of June	2
Second week in June	2
Third week in June	1
Last week in June	8
Latter part of June	2
Any part of June	52

Time and Place for Convention of 1910.

July 99
as follows:

Early part of July	8
First week in July	6
Second week in July	1
First half of July	2
Middle of July	1
Any part of July	81

August	12
September	9
October	2
November	2

"16 members have no choice.

"The following cannot very well be classified:

June or July	5	February or March	1
June and early July	1	Usual time	1
Latter part June, or 1st of July	1	April or May	2
Early summer	1	Most suitable season	1
During spring	1	September or October	1
May or June	5	Either spring or fall	1
July or August	3	Any time during summer	1
		Cool season	1

THE PRESIDENT.—Gentlemen, you have heard the summary of these votes from members at large. It is for you to take action on them.

JOHN F. O'ROURKE, M. AM. SOC. C. E.—I move that the matter be referred to the Board of Direction.

Motion duly seconded.

THE PRESIDENT.—It has been moved and seconded that this matter be referred to the Board of Direction. Is there any discussion on this motion?

C. H. KEEFER, M. AM. SOC. C. E.—Mr. President, I would like to say that if the Society should be inclined to visit us we should be glad to have them come to Ottawa. I think it is some thirteen years since they last visited there. We should like to have them come to Ottawa, our Capital, being the headquarters of the Government, and the Government Engineers being there. I dare say that there is a great deal that would be of interest to them, and a great deal of information that might be obtained in the matter of the different buildings and public works to be visited. There are many things which would be of interest to the members, and at any time that might be convenient—I do not know whether or not it might be convenient to meet there next year, but if they should come next year or the

year after, or even the year after that, we would be very glad to welcome our American cousins and the members of our Society.

F. P. Stearns, Past-President, Am. Soc. C. E.—Mr. President, I have no suggestion to make in regard to the Convention for 1910. I noticed that eleven members voted for Panama. I did not vote for that place this year, because I thought it was rather early, but I wish to make the suggestion at this time, that there might be a trip there the year after, which would give a great deal of pleasure to those who could go.

The best time to visit Panama is in the spring. That is the dry season. The temperature does not vary from one season to another, but the dry season is altogether the best time to go there.

In order to escape the cold weather here, February or March would be the best month. The work there is enormous, and will be in full swing by 1911. I am not in a position to invite you to go there, but I want to call your attention to the advantages of such a trip. I hear of many engineers who say they intend to go there while this great work is in progress. It is not only the great work that is to be seen, but there is to be seen what the American engineer can do in the organization of a strip of territory in the tropics so that work can be carried on in a thoroughly efficient way.

There is a good hotel there on the Pacific side, at Colon—the Hotel Tivoli. I do not know its capacity, but it will hold a very large number. It has a parlor large enough for a meeting room for an audience of this size, and, in fact, I think that in every way the place is well equipped to receive the members of the Society—as many as would be likely to go.

I do not know what arrangements could be made for the trip, but I think they could be made on reasonable terms, especially if we should be able to use some of Uncle Sam's ships.

The climate is comfortable. There is no danger, I believe, to the health of anyone who goes there, and I think it is something we should look forward to at the proper time.

H. H. Quimby, M. Am. Soc. C. E.—Mr. President, I move an amendment to Mr. O'Rourke's motion. I like the suggestion that has been made that a boat be chartered on the Great Lakes to visit from place to place. I think it would be very interesting and very appropriate. I move, in amendment to Mr. O'Rourke's motion, that a clause be added to it that the Board favorably consider that suggestion.

The President.—Is that motion seconded? There does not appear to be any seconder to that amendment. Is there any further discussion on the original motion?

Arthur Hider, M. Am. Soc. C. E.—Mr. Chairman, it seems to me that it is a long time since there has been a Convention in the South, and I think the members of the Society would enjoy what they might

Time and Place for Convention of 1910.

see there; therefore I would suggest that they consider—and that the Board of Direction consider—very seriously the matter of having the next Convention at New Orleans.

I feel assured that the members, and especially those from this section of the country, would see something that they never have seen, and that would be of interest to them.

Mr. Stearns has suggested going to Panama, but if the trip to Panama could be made next year, instead of in 1911, why, from New Orleans, a party could be arranged to go to Panama.

Of course, a great majority of the members could not afford the time to make the visit to Panama, but some of them could. I hope, gentlemen, that the Board of Direction will consider seriously the matter of having one of the conventions down in the South, and I believe that if they decide to have the next one there, in the early part of the season, in April or May, while everything is frozen up here, it would be a delightful trip to members from this section of the country.

THE PRESIDENT.—Any further suggestions or discussion on this motion? The motion is that the question of choosing the place and time for the next Annual Convention be referred to the Board of Direction. All in favor of this will signify by saying "aye"; contrary, "no." The motion is carried.

Report of Board of Direction on Method of Appointing Nominating Committee.

The next item is a report of the Board of Direction on the method of appointing the Nominating Committee. This will be read by the Secretary.

To the Members of the
AMERICAN SOCIETY OF CIVIL ENGINEERS:

At the Annual Meeting of the Society, January 20th, 1909, the following resolution was adopted:

"*Whereas:* The present method of appointing the Nominating Committee of the Society has, in practice, proved to be unrepresentative and unsatisfactory, and

"*Whereas:* Section 2 of Article VII of the Constitution provides that the Board of Direction may prescribe the mode of procedure for appointing the Nominating Committee, therefore be it

"*Resolved:* That the Board of Direction be requested to give further consideration to the whole subject of the method of nominating officers for the Society; and particularly to the present mode of procedure in ascertaining and recording an expression of the corporate membership with regard to the choice of members of the Nominating Committee in the several geographical districts; and to report its conclusions and recommendations to the next Annual Convention of the Society."

The Constitution, Article VII, Section 2, prescribes upon this subject as follows:

"At the Annual Meeting of each year, seven Corporate Members, not officers of the Society, one from each of the geographical districts, shall be appointed by the meeting to serve for two years; who, with

the five living last Past-Presidents of the Society, shall be a committee to nominate officers for the Society.

"The Board of Direction may prescribe the mode of procedure for appointing this Committee, and fill any vacancies occurring.

"The Committee so appointed shall meet at the Annual Convention of the Society, and nominate candidates to fill the offices, named in Article V, so as to provide, with the officers holding over, a Vice-President and six Directors residing in District No. 1, and twelve Directors divided equally, with regard to number and residence, among the remaining districts, Nos. 2, 3, 4, 5, 6, and 7.

"A list of nominees for the offices to be filled at the next Annual Election shall be presented by the Committee to the Board of Direction within ten days after the nominees have been selected."

In addition, Article VII, Section 4, says: "At any time before the first day of December, any ten or more Corporate Members may send to the Secretary additional nominations, signed by such members."

The present mode of procedure for appointing the Nominating Committee is as follows:

In the first part of the month of December, suggestion slips are sent out by the Secretary to all the Corporate Members, the slips merely requesting suggestions of suitable persons for members of the Nominating Committee. Said slips are returnable on or before the Annual Meeting following. The suggested names are then tabulated, and the person receiving the largest number of suggestions in a district is usually appointed by the Meeting on a regularly made motion.

After careful consideration the Board of Direction has adopted a new mode of procedure to go into effect this year as follows:

A Preliminary and a Final Suggestion.

1. A preliminary suggestion slip to be sent out on or before September 1st and returnable on or before November 1st.

2. This suggestion slip to contain an explicit request that the corporate membership will not neglect this important duty, and a statement of the great importance of selecting the right men for the Nominating Committee.

3. All the preliminary suggestions received to be tabulated and the names of the three candidates in each district receiving the highest number of suggestions determined.

4. A final suggestion slip to be sent out at least thirty days before the Annual Meeting, as heretofore, giving the three names in each district resulting from the preliminary suggestions; and also stating that any other eligible member of the Society may be suggested.

5. The final suggestion slips to be then dealt with as at present, viz., the result reported to the Annual Meeting.

By order of the Board of Direction,

CHAS. WARREN HUNT,
Secretary.

July 1st, 1909.

Report on Method of Appointing Nominating Committee.

THE PRESIDENT.—This matter of the method of choosing the Nominating Committee is one that is part of the Board's duty. It was referred to the Board, and, after full consideration, the Board decided to change its method, as has been stated in the report. I do not think that any action can be taken by the members on this report. It is a matter for the Board.

Report of Special Committee on Status of the Metric System.

The next business is the Report of the Special Committeee on the Status of the Metric System. This report was published in the *Proceedings* for October, 1908. Since then we have received a discussion by E. P. North, M. Am. Soc. C. E., who is not present. This discussion can be printed in the *Transactions,* if accepted by the Committee on Publications. It will be in order, therefore, to refer this discussion to that committee. What is your pleasure in the matter, gentlemen?

H. B. SEAMAN, M. AM. SOC. C. E.—Mr. Chairman, is this a final report?

THE SECRETARY.—I think, Mr. President, that some explanation ought to be made. This committee did send in a report, and it was published in the October number of *Proceedings* of last year, and at the Annual Meeting the Secretary understood from the committee that it wished to be discharged, and so stated, and the meeting obligingly discharged the committee. The committee had made no such request in its report, and the Secretary, recognizing his blunder, reported the matter to the Board of Direction, and the Board, using the power vested in it, and using its discretion in the matter, reappointed the committee. So the committee is still in existence, and the report is a progress report.

I do not know that there is any necessity for bringing up this report at this time, but the action of the Annual Meeting was taken without an exact knowledge of the facts, due to the error of the Secretary. The progress report having been published in the *Proceedings,* it does not seem at all necessary to take any action about that, and, if the committee should want to report again, I suppose the matter could come up again.

C. L. STROBEL, M. AM. SOC. C. E.—Is this committee continued indefinitely?

THE SECRETARY.—Yes, sir; the intention, in the appointment of the committee, was that it should be appointed to report from time to time on the "Status of the Metric System."

MR. SEAMAN.—May I ask how many years this is to be continued?

THE SECRETARY.—My recollection is that—I can correct it in the final printing—this committee has been in existence about a year.*

MR. SEAMAN.—How about the previous years?

THE SECRETARY.—It had no previous existence.

* This committee was appointed in January, 1907.—*Secretary.*

MR. STROBEL.—We have had a Committee on the Metric System for many years. It may not have been this committee, but there has been a committee on it. Am I wrong?

THE SECRETARY.—I think you must mean some other society.*

THE PRESIDENT.—Gentlemen, we have the progress report of the Special Committee on Concrete and Reinforced Concrete. This is published in the *Proceedings* for February, 1909, with a statement that "this report has not yet been placed before or accepted by the Society. Its discussion has been made an order of business for the next Annual Convention, July 6th-9th, 1909, and written communications concerning it, received before that date, will be published in *Proceedings*."

Progress Report of Special Committee on Concrete and Reinforced Concrete.

Since then we have received written discussions by John Stephen Sewell, Clarence W. Noble, and S. Bent Russell, Members, Am. Soc. C. E. Have you those discussions here?

Discussion on Report of Special Committee on Concrete and Reinforced Concrete.

THE SECRETARY.—Yes, sir, I have them here.

THE PRESIDENT.—Those discussions can be read, if desired. The report is in your hands for action.

G. F. SWAIN, M. AM. SOC. C. E.—Mr. Chairman, I move that these discussions be read, unless they are too long.

MR. O'ROURKE.—Who is going to pass on the length? There is a motion made, and I will second it, that they be read, if they are not too long.

THE PRESIDENT.—The members can decide that as they hear them.

The Secretary read discussions by Messrs. Sewell, Noble, and Russell.†

THE SECRETARY.—That is all the written discussions I have, sir.

THE PRESIDENT.—Gentlemen, this report is before you, for discussion or action.

MR. O'ROURKE.—Mr. President, about twenty-two years ago I used reinforced concrete in enlarging the sides of one of the old piers built for the Poughkeepsie Bridge, to conform to a new design. I wrote a paper on the subject, and it is in the *Transactions* for 1887. Ever since that time I have been using concrete, more or less reinforced. Some of it is being used now in the same manner that buckle-plates are used. It is used to take the place of reinforced units—we will call them—of construction.

Now, I have no faith whatever in reinforced concrete as a unit material. It is a combination, and I object to any report on reinforced concrete, taking it as something like steel or wood, or something that you can depend on, and know that you are getting results which will correspond with data that you have on the subject. There are no two

* A Special Committee to "report a form of memorial to Congress, in furtherance of the adoption of the metric standards * * *," was appointed in February, 1877, and made reports in April and November, 1877. On February 6th, 1878, the following resolution was adopted: "Resolved, that the further consideration of the metric system of weights and measures be indefinitely postponed."—*Secretary.*

† Will be published in the September, 1909, number of *Proceedings.*

Discussion on Report of Special Committee on Concrete and Reinforced Concrete.

mixtures of concrete alike, unless each has been skilfully mixed, no matter what the material is, and there has been no proof yet that the large temperature strains which exist on the surface of reinforced concrete, or of any concrete, are going to stand the test of time.

Now, you can have all the report you like, and you need not concur with the minority report, which says that a vote does not make reinforced concrete what you think it is; and all I have to say is that the American Society of Civil Engineers is departing from its old principle of standing by the things that they know, and not depending on the things that they do not know. Now, that is my opinion of those reports.

J. R. WORCESTER, M. AM. SOC. C. E.—Mr. President, if it is in order, I would like to present a little discussion of the minority report. I presume that is also under consideration.

THE PRESIDENT.—Yes, sir; won't you come forward and face the audience, please?

Mr. Worcester read his discussion.*

A MEMBER.—Mr. President, in order to carry this discussion along intelligently, permit me to suggest that you take up the sections of the report for discussion, to prevent any confusion of ideas by going along the entire report.

THE PRESIDENT.—I think that would have to come up in the form of a motion. This report has not been accepted by the Society. It is merely published for discussion, as I understand it. The publication is headed: "This report has not yet been placed before or accepted by the Society. Its discussion has been made an order of business for the next Annual Convention." That is, for this meeting. Therefore, it is for you, gentlemen, to determine what you will do with it.

L. J. JOHNSON, M. AM. SOC. C. E.—Mr. President, it seems pretty clear that this matter has had very careful consideration by a committee which this Society may very well trust.

As I understand it, all that is asked is that this report be accepted precisely the same as the progress reports on the tests of cements have been accepted in the past. It seems to me that nobody supposes that the committee on the testing of cement has yet reached a conclusion not subject to change—to say nothing of the finish of their investigations at the time they reported. It seems, then, as a parallelism with this cement testing, that it is only proper and natural that this carefully thought out body of results should be accepted and put on record where it can be seen.

I do not think anybody will misunderstand them. That is, I do not think serious people will misunderstand them, and reckless people will not be governed by them. Personally, I would not say that I would endorse every conclusion. I do not think I would agree with all

* Will be published in the September, 1909, number of *Proceedings*.

that the committee says on cement testing. I would not feel bound to accept every one of their recommendations, but, as a collection of data embodying the most careful present-day practice in the use of reinforced concrete, it seems to me that the Society would be losing an opportunity by not putting this matter in its permanent archives.

It is very closely akin to the standing recommendations made by the practitioners of Europe, especially the German practice, where I take it that reinforced concrete has been carried to a degree of practicability that this country cannot surpass, if it can be equaled here.

It seems to me that it is very proper that this report be accepted, and I make that motion, Mr. President.

Motion duly seconded.

THE PRESIDENT.—Gentlemen, you have heard the motion. Any discussion upon it? The motion is that the majority report be accepted. A motion is before the house now.

R. W. LESLEY, ASSOC. AM. SOC. C. E.—To be taken as a report of progress?

MR. JOHNSON.—To be taken as a report of progress. I also suggest that the committee be continued, and that the report be published.

THE PRESIDENT.—It has been suggested that it would be better that the motion should read that the report be received and ordered printed in the *Transactions*.

MR. JOHNSON.—That is satisfactory to me.

THE PRESIDENT.—Would you include the discussions that have just been read?

MR. JOHNSON.—I should think so, if the other discussions stand.

THE PRESIDENT.—And the committee to be continued?

MR. JOHNSON.—Yes, sir.

A MEMBER.—Does it include the minority report also?

THE PRESIDENT.—No, sir.

MR. G. F. SWAIN.—Mr. Chairman, I sympathize entirely with the views expressed by Professor Johnson, but I think the minority report ought to be printed, if only as a discussion, in line with the other discussions that have been received as to the majority report of the committee.

THE SECRETARY.—The Secretary also understands that the discussions as submitted should be submitted to the committee, and that the committee should be given an opportunity to write anything it pleases in connection with this original report, in order to make the matter complete in the final report, in the *Transactions*.

RICHARD L. HUMPHREY, M. AM. SOC. C. E.—I also suggest that other discussions which come in later follow the same course of procedure that the Secretary has suggested. One of the objects in presenting this report was to draw out discussion which would be helpful to the committee, and I wish to say that I do not believe that any of

Discussion on Report of Special Committee on Concrete and Reinforced Concrete.

the discussion which has been heretofore received has been of a constructive character. There has been nothing in the discussions, nor are there any suggestions, in the light of bettering the report of the committee. The discussions which the committee particularly desired to have are those which would suggest improvements in the report itself, and I believe the purpose will be served if the discussions, as they come in, be referred to the committee and published.

I would like to say, also, that there are some typographical errors in the report, and these the committee would like to have corrected; for instance, there is one in which the angle, the bent-up angle, reads 75°, whereas it should read, 45°, just as Captain Sewell suggested.

It may be a matter of interest to state that this committee was by resolution asked to co-operate with Committees of the Maintenance of Way Association, and of the Society for Testing Materials, and it might be interesting to know that of the total membership of that committee—some thirty-three—twenty-seven were for the report, four were against it, and two did not vote; and, of the twenty-seven members who voted for the report, twenty-five were members of this Society.

THE PRESIDENT.—Are you ready for the question? Those in favor of the motion will signify it by saying "aye"; contrary, "no." The motion is carried. The Secretary has a paper which he desires to read.

Letter from San Francisco Association, Am. Soc. C. E.

THE SECRETARY.—The Secretary forgot to read the following letter, when the matter of the time and place for the Annual Convention was up:

"SAN FRANCISCO, CAL., June 25, 1909.

"CHARLES WARREN HUNT, Secretary,
"Am. Soc. C. E.,
"220 W. 57th St.,
"New York City.

"DEAR SIR:

"At the regular bi-monthly meeting of the San Francisco Association of Members, the matter of the Annual Convention of 1910 was discussed at length, and it was unanimously resolved that—inasmuch as the Pacific Coast, and particularly this City, as the geographical, industrial, and financial center thereof, appears to present superior attractions for such an occasion in the way of important engineering works, and evidence of rapid material development; and, inasmuch as San Francisco, America's eighth city in population and wealth, famed the world over for its generous and well-ordered hospitality, has already, three years after utter physical ruin, eclipsed its former self with larger and finer buildings and better and cleaner streets, and will by next year present facilities for varied entertainment unrivaled by those of any other place—the Society should be cordially invited to hold its Annual Convention in this City in the autumn of 1910. In extending this invitation, we respectfully invite your attention to the following works, constituting a few of the attractive features that would attend an excursion of engineers to San Francisco:

"In San Francisco and its immediate suburbs, in addition to the latest types and methods of building construction and equipment, and the latest developments in connection with public service, are to be seen the latest improvements in dock construction, the immense dry-dock and ship-building plants at Mare Island and Hunter's Point, and the Union Iron Works (where the U. S. Battleship *Oregon* was built), the electrification of the extensive suburban lines of the Southern Pacific Company, the electric power plants at Martin Station, South San Francisco, with its three 5 400-h.p. gas-engine units, and, at Oakland, where a single Curtis turbine unit of 9 000-kw. capacity is in successful operation.

"Short excursions may be arranged to include with convenience visits to the following points of interest: The plants of the Pacific Gas and Electric Company at Electra, with a capacity of 2 000 kw., under 1 460-ft. head; at de Sabla, with 14 000-kw. capacity under 1 530-ft. head; at Centerville with a 5 500-kw. Francis turbine operating under 590-ft. head (the highest in the world); the high-tension (60 000 volts) circuits from Electra and de Sabla are, respectively, 140 and 250 miles long;

"The plant of the Great Western Power Company near Oroville, a 60 000-h.p. installation with about 160 miles of steel towers carrying two transmission circuits with a capacity of 100 000 volts each;

"Gold-dredging work near Oroville and Folsom with twenty-four dredgers representing an investment of $7 500 000.00 in machinery, working gravel which produces 6 to 10 cents per cubic yard at as high a rate as 300 000 cu. yd. per month for the largest dredgers at a cost of less than 2 cents per cubic yard, the annual output being $10 000 000.00;

"Smelting works at Selby's, near San Francisco; the copper smelters at Kennett and Coram, and the celebrated electric furnace at Heroult in Shasta County;

"Three large Portland cement plants within fifty miles of San Francisco; the immense petroleum and asphalt industries of Kern, Fresno, Santa Barbara, Los Angeles, and Orange Counties.

"Farther removed from San Francisco, but easily accessible en route, may be mentioned the Owens River Aqueduct construction work in Southern California, the Klamath and Orland irrigation works in Northern California, and the Yuma and Roosevelt Dams.

"San Francisco is the center of the many beautiful pleasure and scenic resorts along the coast and in the mountains of California. These points of interest are so well-known that further reference to them need not be made.

"San Francisco's equipment, both natural and artificial, for entertaining guests is certainly unexcelled; the weather is fair and balmy, twenty-four hours of every day, during the Autumn months, being a comfort and delight. Among scores of good-sized, first-class hostelries and many brilliant cafés, this City has three of the largest and most modern hotels in the world, steel frame, fire-proof construction, the best of service, and very moderate rates. The many theaters are of beautiful architecture, steel-frame, fire- and earthquake-proof construction.

"We respectfully submit that in point of time it is highly

Letter from San Francisco Association, Am. Soc. C. E.

opportune that the Society favor San Francisco with a visit, and we can assure the members that they will have set before them such a feast of reason and observation and flow of soul that they will of one accord declare that convention to be the most successful ever held by the Society.

"We send you under separate cover, with our compliments, birds-eye views of San Francisco, taken shortly after the earthquake and early this year, respectively, which will afford some idea of what has been done to reclaim the vast area of ashes and ruins that resulted from the fire of 1906. We also send you a small map of California on which are indicated the locations of some of the engineering works of interest in this vicinity.

"Trusting the one hundred and ten members of our Association, together with the balance of the three-quarters of a million people residing in San Francisco and its immediate suburbs will have the pleasure of entertaining the Society during the Autumn of 1910, we are,

"Sincerely yours,
"San Francisco Association of Members, Am. Soc. C. E.
"LUTHER WAGONER,
"President.

"E. L. THURSTON, JR.,
"Secretary."

Letters from Mayor and Citizens' Business League of Milwaukee.

I also have an invitation from the Mayor of Milwaukee, and from the Citizens' Business League of Milwaukee, to hold the next Convention in that city.

Announcements.

I also have to announce, in the regular order, the following elections and transfers by the Board of Direction on the 1st of July.*

It may be interesting to state that there are eighty-five names on this list, and that they come from twenty-nine of the subdivisions of the United States, three from the subdivisions of Canada, two from Mexico, one from New Zealand, one from England, one from Panama, and one from the Philippine Islands.

I also have to announce, Mr. President, with regret, the following deaths.†

The programme says, Mr. President, that to-morrow evening there will be a meeting in this hall for the discussion of professional topics, which have been suggested.

Discussion on Change of Programme.

MR. O'ROURKE.—Mr. Chairman, I was approached by a number of ladies, and several of their admirers, and requested to say that it would please them very much if we would postpone the topics until Friday night, and have the ball to-morrow night, instead of Friday night. So I move that we substitute for the discussion of topics to-morrow night the programme that was arranged for Friday night, the Reception and Ball, having on Friday night that which was intended for to-morrow night.

* See pages 270-272.

† See page 272.

Motion duly seconded by Mr. Hatton.

THE PRESIDENT.—It is before the house, and open for discussion.

MR. WORCESTER.—Mr. President, it seems to me that that is doing great justice to the ladies and their admirers, but hardly fair justice to the members of the Society who have interest in the professional topics of the Society. I think that a great many have made their plans to attend the meetings of the Convention, and have not made plans to stop over for the social portion on Friday, and I think it would be hardly fair to them to put them off in this way.

S. WHINERY, M. AM. SOC. C. E.—Mr. Chairman, would there be any objection to holding a meeting for this discussion to-morrow afternoon, in order to accommodate the members who wish to go away early? I merely ask for information.

THE PRESIDENT.—I think the meeting can determine that.

MR. O'ROURKE.—Mr. Chairman, talking to the motion, I think that these conventions are intended for two things: first, the discussion of the same things that we are discussing all the year, and next the meeting with one another which we get once a year. Now, a great many people are going on Friday, and will not be here on Friday night, and it is all right to say that you can discuss, and you can work, and you can grind, but it is said that Jack is a dull boy unless he gets a little play once in awhile, and I think this is a case of that kind; therefore, in justice to our folks who come here to enjoy themselves, I think we ought to adjourn our business meeting and give the ladies their innings to-morrow night. I do not see what harm there is in having that Reception to-morrow night instead of waiting until most of the people have gone home; anyway, this is the first Convention at which I ever knew the Ball to be postponed until the last night, and therefore, as the members can arrange to discuss anything they like at any time they wish, I think, in justice to those who are not members themselves, but are the members' bosses, that we should let them have their innings to-morrow night.

THE SECRETARY.—Mr. President, would there be any objection to having this meeting for the discussion of professional topics to-morrow afternoon?

A MEMBER.—No.

MR. SEAMAN.—To-morrow morning?

THE SECRETARY.—That was not my question.

MR. SEAMAN.—I think that we have been very seriously criticized abroad because we do not have as much professional discussion as we had in former years, and, while I have the deepest sympathy for Mr. O'Rourke's proposition and would be glad to vote for it, I think we should let other things step aside for professional discussion. We have had all day given up to enjoyment, and plan to give all day to enjoyment to-morrow, and now we are asked to give up to-morrow evening

Discussion on Change of Programme.

for enjoyment, and still no discussion. I should like to have the discussion to-morrow afternoon, if it cannot take place to-morrow morning.

THE SECRETARY.—There is no reason why it cannot take place all day, so far as I know.

MR. SEAMAN.—That is what we ought to have. I do not think we have enough of it.

MR. WHINERY.—May I ask further—the Secretary is better informed than most of us about the programme, on both social and business matters—what time to-morrow would be most convenient, with reference both to pleasure and to business, to hold this session, if one is decided on?

THE SECRETARY.—I do not think it makes any very great difference, Mr. President, but it seems to me that the afternoon would be the better time. There is nothing to interfere with it in the afternoon.

THE PRESIDENT.—What hour would you suggest?

THE SECRETARY.—I should say two o'clock.

MR. WHINERY.—In order to bring it before the meeting, I move that a meeting of the Convention for the discussion of engineering topics be held at two o'clock to-morrow afternoon.

THE PRESIDENT.—Mr. Whinery, there is a motion before the house.

MR. O'ROURKE.—I accept the amendment, Mr. President.

THE PRESIDENT.—The motion then, as I understand it, is that, instead of holding the meeting for the discussion of professional topics to-morrow evening at eight o'clock, it will be held at two o'clock to-morrow afternoon, and that the time for the Convention Ball will be changed from Friday evening to Thursday evening. Are you ready for the question? Those in favor of the motion will signify it by saying "aye"; contrary, "no." The motion is carried.

Announcement.

THE SECRETARY.—I am reminded that I should announce to you that the certificates for the purchase of your return tickets are ready for you, having been properly viséd by the railroad man; they can be obtained at the Secretary's office at any time.

THE PRESIDENT.—This ends all the business for the evening. A motion to adjourn will be in order.

Motion to adjourn duly seconded; unanimously carried.

Adjourned.

FORTY-FIRST ANNUAL CONVENTION

EXCURSIONS AND ENTERTAINMENTS

Excursions had been arranged by the Local Committee to the Summit of Mt. Washington, to Crawford Notch, and Bemis, and to the Profile House. Owing to weather conditions there was a slight change in the programme, and the large party which went to the Summit on the afternoon of July 6th were fortunate in securing a clear view. Nearly every one present made the trips above mentioned, and many also visited "The Flume" and drove to the Summit of Mt. Willard. These trips, together with the delightful short walks and drives, were much enjoyed.

A well-attended and enjoyable Reception, with dancing, was held on the evening of Thursday, July 8th.

THE ATTENDANCE AT THE FORTY-FIRST ANNUAL CONVENTION

The following 177 members were in attendance. There were also present 145 ladies and others of the families of members.

Aiken, W. A..Philadelphia, Pa.
Allen, C. Frank...Boston, Mass.
Allen, C. M....Worcester, Mass.
Allen, Kenneth..New York City
Alvord, J. W.....Chicago, Ill.
Archbald, James..Scranton, Pa.
Arnold, W. H..New York City
Atwood, W. G.Indianapolis, Ind.
Auryansen, F..New York City

Babcock, W. S..New York City
Baldwin, F. H..Bayonne, N. J.
Ball, C. B.....Chicago, Ill.
Barbour, F. A.....Boston, Mass.
Bates, Onward.....Chicago, Ill.
Bennett, L. J....Buffalo, N. Y.
Bensel, J. A....New York City
Beugler, E. J....New York City
Bott, J. B.....Greensburg, Pa.
Bowers, George...Lowell, Mass.
Brackett, Dexter..Boston, Mass.
Breed, C. B.....Lynn, Mass.
Brewer, Bertram.Waltham, Mass.
Brooks, Fred.....Boston, Mass.
Brooks, J. P.....Urbana, Ill.
Brown, W. P.,
South Orange, N. J.
Burden, James.....Troy, N. Y.
Carpenter, C. E..Yonkers, N. Y.
Chase, R. D..New Bedford, Mass.
Churchill, C. S....Roanoke, Va.
Clapp, O. F..Providence, R. I.
Codwise, E. B..Kingston, N. Y.
Conkling, C. C..Buffalo, N. Y.
Connor, E. H.,
Leavenworth, Kans.
Cook, J. H.....Passaic, N. J.
Cotton, J. P....Newport, R. I.
Craven, Alfred..Yonkers, N. Y.
Crocker, H. S.....Denver, Colo.
Cummings, R. A..Pittsburg, Pa.
Cunningham, A. O.,
St. Louis, Mo.

Davis, C. E....Kingston, N. Y.
Davis, J. L.....New York City
Davis, W. R.....Albany, N. Y.
Davison, G. S....Pittsburg, Pa.
Dean, A. W....Nashua, N. H.
Dean, Luther....Taunton, Mass.
Doten, L. S..Washington, D. C.
Douglas, B....Grosse Isle, Mich.
Dutton, C. H..Waltham, Mass.

Elliott, M..Culebra, Canal Zone
Ellis, J. W...Woonsocket, R. I.

Eltinge, O. L..Kingston, N. Y.
Emerson, G. C....Boston, Mass.

Farnum, L. N..New York City
Flad, Edward...St. Louis, Mo.
Follett, W. W....El Paso, Tex.
Ford, W. G....Brooklyn, N. Y.
Foss, F. E.....Pittsburg, Pa.
Francis, G. B...New York City
French, A. H...Brookline, Mass.

Gardiner, F. W..Yonkers, N. Y.
Gaston, L. P..Somerville, N. J.
Gerber, Emil.....Pittsburg, Pa.
Green, B. R..Washington, D. C.
Greiner, J. E....Baltimore, Md.

Haines, H. S...New York City
Hale, R. A....Lawrence, Mass.
Hammond, G. T.New York City
Hatton, T. C..Wilmington, Del.
Hazen, J. V.....Hanover, N. H.
Henderson, J. T.,
Hartford, Conn.
Herbert, H. M.,
Bound Brook, N. J.
Hider, Arthur.Greenville, Miss.
Hill, W. R.....Albany, N. Y.
Hodgdon, F. W...Boston, Mass.
Hogue, Chester J..Boston, Mass.
Holden, C. A...Hanover, N. H.
Horton, Horace E..Chicago, Ill.
Howard, C. P..Cleveland, Ohio
Humphrey, R. L.,
Philadelphia, Pa.
Hunt, Charles Warren,
New York City
Hyde, C. G.....Berkeley, Cal.

Johnson, L. J.Cambridge, Mass.
Johnson, T. H....Pittsburg, Pa.
Johnson, W. S....Boston, Mass.
Jordan, E. C.....Portland, Me.

Keefer, C. H.,
Ottawa, Ont., Canada
Kimball, G. A....Boston, Mass.
King, W., Jr...New York City
Knickerbacker, John,
Troy, N. Y.
Koon, R. E...Kansas City, Mo.

Lesley, R. W..Philadelphia, Pa.
Lewis, E. W..New Haven, Conn.
Lewis, N. P....Brooklyn, N. Y.
Lindenthal, G..New York City
Loomis, Horace..New York City
Loweth, C. F.....Chicago, Ill.
Lowinson, Oscar.New York City
Lynch, M. L.....Tyler, Tex.

McKenzie, T. H.,
Southington, Conn.
Manley, Henry....Boston, Mass.
Manley, L. B.....Boston, Mass.
Miller, C. H..Little Rock, Ark.
Miller, R. P...New York City
Mills, C. M....Philadelphia, Pa.
Morrow, J. J.,
Washington, D. C.
Morse, W. P....Newton, Mass.
Moses, J. C..Cambridge, Mass.
Mott, W. E....Brookline, Mass.
Murtaugh, M. M.,
Salt Lake City, Utah

Ockerson, J. A..St. Louis, Mo.
O'Rourke, J. F..New York City
Ostrander, J. E..Amherst, Mass.
Owen, James..Montclair, N. J.
Owen, K. D...Montclair, N. J.

Pegram, G. H..New York City
Perry, F. W....Brooklyn, N. Y.
Polk, W. A.....New York City
Powers, J. A.....Troy, N. Y.
Putnam, G. R.,
Washington, D. C.

Quimby, H. H.Philadelphia, Pa.
Quincy, C. F...New York City

Richardson, T. F.,
Cambridge, Mass.
Ricker, G. A....Buffalo, N. Y.

Ricketts, P. C.....Troy, N. Y.
Ridgway, R.,
Poughkeepsie, N. Y.
Riggs, M. J.....Toledo, Ohio
Rollins, J. W., Jr.,
Boston, Mass.

Sax, P. M....Philadelphia, Pa.
Scammell, J. K.,
St. John, N. B., Canada
Seaman, H. B..New York City
Skinner, F. W..New York City
Skinner, J. F..Rochester, N. Y.
Small, G.....Waltham, Mass.
Smith, E. M...New York City
Smith, J. Waldo..New York City
Smith, M. H.,
White Plains, N. Y.
Spear, W. E....Brooklyn, N. Y.
Spofford, C. M..New York City
Staniford, C. W.,
Brooklyn, N. Y.
Stearns, F. P....Boston, Mass.
Stearns, R. H..New York City
Stowe, H. C....Brooklyn, N. Y.
Strobel, C. L.....Chicago, Ill.
Stuart, Francis Lee,
Essex Fells, N. J.
Sutton, Frank,
Washington, D. C.
Swain, G. F.....Boston, Mass.
Swensson, Emil...Pittsburg, Pa.

Talbot, A. N.....Urbana, Ill.
Thackray, G. E..Johnstown, Pa.
Thomas, C. D..Brooklyn, N. Y.
Thompson, F.....Boston, Mass.
Thompson, S. C..New York City
Thompson, S. E.,
Newton Highlands, Mass.
Thorn, A. W....Buffalo, N. Y.
Tighe, J. L.....Holyoke, Mass.
Tinkham, S. E....Boston, Mass.
Tripp, O. H.....Rockland, Me.

Van Ornum, J. L.St. Louis, Mo.
Verveer, E. L..New York City

Wagner, J. C..Philadelphia, Pa.
Watt, D. A.....Albany, N. Y.
Webster, G. S..Philadelphia, Pa.
Wendt, E. F.....Pittsburg, Pa.
Wentworth, C. C..Roanoke, Va.
Weston, R. S..Brockton, Mass.
Whinery, S..East Orange, N. J.
White, J. G....New York City
White, L.....New York City
Whitney, F. O....Boston, Mass.
Williams, G. S.,
Ann Arbor, Mich.
Wilson, P. H..Philadelphia, Pa.
Wing, C. B.,
Stanford University, Cal.
Woodbury, C. J. H.,
Boston, Mass.
Woodruff, E. L.,
Los Angeles, Cal.
Woods, H. D.,
West Newton, Mass.
Worcester, J. R..Boston, Mass.
Wyckoff, C. R., Jr.,
Brooklyn, N. Y.

ANNOUNCEMENTS

The House of the Society is open from 9 A. M. to 10 P. M., every day, except Sundays, Fourth of July, Thanksgiving Day, and Christmas Day.

MEETINGS

September 1st, 1909.—8.30 P. M.—At this meeting two papers will be presented for discussion, as follows: "Concrete Piles," by Howard J. Cole, M. Am. Soc. C. E., and "Hydro-Electric Power in Canada," by Cecil B. Smith, M. Am. Soc. C. E.

These papers were printed in *Proceedings* for May, 1909.

September 15th, 1909.—8.30 P. M.—Two papers will be presented for discussion, as follows: "The Use and Care of the Current Meter, as Practiced by the United States Geological Survey," by John C. Hoyt, Assoc. M. Am. Soc. C. E., and "Walnut Lane Bridge, Philadelphia," by George S. Webster and Henry H. Quimby, Members, Am. Soc. C. E.

These papers are printed in this number of *Proceedings*.

October 6th, 1909.—8.30 P. M.—At this meeting two papers will be presented for discussion, as follows: "A Review of Chicago Paving Practice," by P. E. Green, Assoc. M. Am. Soc. C. E., and "The Purification of the Water Supply of Steelton, Pennsylvania," by James H. Fuertes, M. Am. Soc. C. E.

These papers are printed in this number of *Proceedings*.

October 20th, 1909.—8.30 P. M.—A paper by Chester W. Larner, Esq., on "Characteristics of Modern Hydraulic Turbines," will be presented for discussion.

This paper is printed in this number of *Proceedings*.

PAPERS AND DISCUSSIONS

The first and second volumes of *Transactions* for 1909 (Volumes LXII and LXIII) have been issued. Two additional volumes will be issued during the year.

It is hoped that members and others who take part in the discussion of the papers presented will revise their remarks promptly, and that all written communications from those who cannot attend the meetings will be sent in at the earliest possible date after the issue of a paper in *Proceedings*. The issue of volumes of *Transactions* is dependent on the closing of discussions, and the co-operation of the membership will now be more necessary in this matter than heretofore, because four volumes are to be issued during the year, instead of two, and, to accomplish this, a definite date of issue for each has been established. It is expected that the third and fourth volumes for 1909 will be issued on September 30th and December 31st, respectively.

All papers accepted by the Publication Committee are classified by the Committee with respect to their availability for discussion at meetings.

Papers, which, from their general nature, appear to be of a character suitable for oral discussion, will be published as heretofore in *Proceedings*, and set down for presentation to a future meeting of the Society, and, on these, oral discussion, as well as written communications, will be solicited.

All papers which do not come under this heading, that is to say, those which, from their mathematical or technical nature, in the opinion of the Committee, are not adapted to oral discussion, will not be scheduled for presentation to any meeting. Such papers will be published in *Proceedings* in the same manner as those which are to be presented at meetings, but written discussions, only, will be requested for subsequent publication in *Proceedings* and with the paper in the volumes of *Transactions*.

PRIVILEGES OF ENGINEERING SOCIETIES EXTENDED TO MEMBERS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

Members of the American Society of Civil Engineers will be welcomed by the following Engineering Societies, both to the use of their Reading Rooms and at all Meetings:

American Institute of Mining Engineers, 29 West Thirty-ninth Street, New York City.

Associação dos Engenheiros Civis Portuguezes, Lisbon, Portugal.

Australasian Institute of Mining Engineers, Melbourne, Victoria, Australia.

Boston Society of Civil Engineers, 715 Tremont Temple, Boston, Mass.

Brooklyn Engineers' Club, 197 Montague Street, Brooklyn, N. Y.

Canadian Society of Civil Engineers, 413 Dorchester Street, West, Montreal, Que., Canada.

Civil Engineers' Society of St. Paul, St. Paul, Minn.

Cleveland Engineering Society, 718 Caxton Building, Cleveland, Ohio.

Cleveland Institute of Engineers, Middlesbrough, England.

Colorado Association of Members, Am. Soc. C. E., H. J. Burt, Secy., 235 Equitable Building, Denver, Colo.

Engineers' and Architects' Club of Louisville, Ky., 303 Norton Building, Fourth and Jefferson Streets, Louisville, Ky.

Engineers' Club of Baltimore, Baltimore, Md.

Engineers' Club of Central Pennsylvania, Corner Second and Walnut Streets, Harrisburg, Pa.

Engineers' Club of Minneapolis, 17 South Sixth Street, Minneapolis, Minn.

Engineers' Club of Philadelphia, 1317 Spruce Street, Philadelphia, Pa.

Engineers' Club of St. Louis, 3817 Olive Street, St. Louis, Mo.

Engineers' Club of Toronto, 96 King Street, West, Toronto, Ont., Canada.

Engineers' Society of Western Pennsylvania, 803 Fulton Building, Pittsburg, Pa.

Institute of Marine Engineers, 58 Romford Road, Stratford, London, E., England.

Institution of Engineers of the River Plate, Buenos Aires, Argentine Republic.

Institution of Naval Architects, 5 Adelphi Terrace, London, W. C., England.

Junior Institution of Engineers, 39 Victoria Street, Westminster, S. W., London, England.

Koninklijk Instituut van Ingenieurs, The Hague, The Netherlands.

Louisiana Engineering Society, 604 Tulane-Newcomb Building, New Orleans, La.

Memphis Engineering Society, Memphis, Tenn.

Midland Institute of Mining, Civil and Mechanical Engineers, Sheffield, England.

Montana Society of Engineers, Butte, Montana.

North of England Institute of Mining and Mechanical Engineers, Newcastle-upon-Tyne, England.

Oesterreichischer Ingenieur- und Architekten-Verein, Eschenbachgasse 9, Vienna, Austria.

Pacific Northwest Society of Engineers, 617-618 Pioneer Building, Seattle, Wash.

Rochester Engineering Society, Rochester, N. Y.

Sachsischer Ingenieur- und Architekten-Verein, Dresden, Germany.

Sociedad Colombiana de Ingenieros, Bogota, Colombia.

Societe des Ingenieurs Civils de France, 19 Rue Blanche, Paris, France.

Society of Engineers, 17 Victoria Street, Westminster, S. W., London, England.

Svenska Teknologforeningen, Brunkebergstorg 18, Stockholm, Sweden.

Tekniske Forening, Vestre Boulevard 18–1, Copenhagen, Denmark.

Western Society of Engineers, 1737 Monadnock Block, Chicago, Ill.

SEARCHES IN THE LIBRARY

In January, 1902, the Secretary was authorized to make searches in the Library, upon request, and to charge therefor the actual cost to the Society for the extra work required. Since that time many searches have been made, and bibliographies and other information on special subjects furnished.

The resulting satisfaction, to the members, who have made use of the resources of the Society in this manner, has been expressed frequently, and leaves little doubt that, if it were generally known to the membership that such work would be undertaken, many would avail themselves of it.

The cost is trifling compared with the value of the time of an engineer who looks up such matters himself, and the work can be performed quite as well, and much more quickly, by persons familiar with the Library.

In asking that such work be undertaken, members should specify clearly the subject to be covered, and whether references to general books only are desired, or whether a complete bibliography, involving search through periodical literature, is desired.

In reference to this work, the Appendix* to the Annual Report of the Board of Direction for the year ending December 31st, 1906, contains a summary of all searches made to that date.

LOCAL ASSOCIATIONS OF MEMBERS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

San Francisco Association

(Abstract of Minutes of Meeting)

June 18th, 1909.—The twenty-fifth regular meeting of the San Francisco Association of Members, Am. Soc. C. E., was held; Luther Wagoner, President, in the chair; E. T. Thurston, Jr., Secretary; and present, also, 35 members and guests.

It was voted to hold the weekly informal luncheons hereafter at 12.15 P. M. every Wednesday, at the Bismarck Café.

The Secretary read a communication from the Secretary of the Society calling attention to the fact that the publication of discussions in the *Proceedings* of the Association would prevent their subsequent publication in the *Transactions* of the Society, and suggesting the advantage of forwarding discussions directly to the Society, thereby insuring greater publicity. It was agreed that such would be the procedure in the future.

* *Proceedings*, Vol. XXXIII, p. 20 (January, 1907).

The Secretary reported the acquisition of three new members since the last meeting.

Mr. William George, manager of a local cement factory, was introduced as one of the guests of the evening, and invited the Association to visit the works of his company. The invitation was accepted, and the President and Secretary were directed to represent the Association in making the necessary arrangements.

The matter of the Annual Convention of the Society for 1910 was discussed, and a motion to invite the Society to hold its next Convention in San Francisco was carried unanimously.

The paper by W. S. McFetridge, M. Am. Soc. C. E., entitled "Some Extensive Railroad Surveys, and Their Cost per Mile," was discussed by Messrs. G. L. Dillman, W. C. Hammatt, and F. Riffle.

The paper by Howard J. Cole, M. Am. Soc. C. E., entitled "Concrete Piles," was discussed by Messrs. L. J. Mensch, M. L. Tower, J. H. Wise, and S. G. Hindes.

The paper by D. A. Usina, Assoc. Am. Soc. C. E., entitled "Copyright in Drawings of a Technical Character," was discussed by L. J. Mensch, M. Am. Soc. C. E.

Adjourned.

ACCESSIONS TO THE LIBRARY

(From May 11th to July 31st, 1909)

DONATIONS*

POCKET-BOOK OF USEFUL FORMULÆ AND MEMORANDA FOR CIVIL, MECHANICAL AND ELECTRICAL ENGINEERS.

By *Sir* Guilford L. Molesworth and Henry Bridges Molesworth. Twenty-Sixth Edition, Revised and Enlarged with an Electrical Supplement by Walter H. Molesworth. Leather, 5 x 3 in., illus., 9 + 901 pp. London, E. & F. Spon, Ltd.; New York, Spon & Chamberlain, 1908. $2.00 net.

In the preface to the original edition, dated 1862, this work is stated to be a concise and comprehensive manual of data for the various and rapid calculations so constantly needed in the ordinary operations of the engineer. The preface of the present edition states that the authors have made very considerable additions and alterations, suggested by their progressive experience, by the advance of engineering science, and by professional criticism, the substitution of new matter for that which has become obsolete and the condemnation of those portions unnecessarily diffuse, enabling them to do this without increase in bulk.

PURE MILK AND THE PUBLIC HEALTH.

A Manual of Milk and Dairy Inspection. By Archibald Robinson Ward, With Two Chapters by Myer Edward Jaffa. Cloth, 9 x 6 in., illus., 13 + 218 pp. Ithaca, N. Y., Taylor & Carpenter, 1909. $2.00 net.

The author has attempted, he states in his preface, the task of assembling, for the information of the health officer and others directly concerned in the crusade for better milk, the facts essential in coping with the difficulties encountered in carrying on efficient and comprehensive measures for milk and dairy inspection. Each chapter is followed by a list of references bearing on the subjects discussed, and referred to in the text by numbers in parentheses. The Contents are: The Contamination of Milk; Changes in Milk Caused by Bacteria; Epidemic Diseases Transmitted by Milk; Bovine Tuberculosis and Other Cattle Diseases; Municipal Sanitary Control of Milk; Pasteurization of Milk; Microscopic Tests of Milk; Bacteriological Examination of Milk; Certified Milk; The Analysis of Milk; Adulteration of Milk; Figures in Text; Appendix A, Dairy Ordinance of Berkeley, Cal.; Appendix B, Chicago Tuberculin Test Ordinance; Appendix C, Duluth Milk Ordinance; Appendix D, Blank for Reporting Existence of Infectious Diseases; Appendix E, Poster for Dairy Barns; Index.

THE REINFORCED CONCRETE POCKET BOOK.

Containing Useful Tables, Rules and Illustrations for the Convenient Design, Rational Construction and Ready Computation of Cost of Reinforced Concrete Girders, Slabs, Footings, Columns, Buildings, Retaining Walls, Tanks, Grain Elevators, Coal Bins, Water Pipes, Sewers, Dams, Bridges, Smoke Stacks, Piles, etc., etc. By L. J. Mensch, M. Am. Soc. C. E. Leather, 6¼ x 4¼ in., illus., 216 pp. San Francisco, The Author, 1909. $10.00.

The object of this pocket book is to place before the public, in as concise a form as possible, practical information in regard to reinforced concrete construction, and the author also expresses the hope that the book will promote the use of reinforced concrete and be the medium of its standardization. It is not a textbook, he says, and a good knowledge of mechanics and of building construction is indispensable to its rational use. The tables, of which there are a great number, are said to have been very carefully calculated and checked repeatedly by slide-rule, and most of them, having been calculated some years ago, have also been thoroughly tested by practical application. The preface further states that the compact form of the book has compelled the author to confine the information given to those subjects which cannot readily be found in other books and to those applications of reinforced concrete which are most used.

* Unless otherwise specified, books in this list have been donated by the publisher.

PRACTICAL ALTERNATING CURRENTS AND ALTERNATING CURRENT TESTING.

By Charles F. Smith. Third Edition, Revised and Enlarged. Cloth, 8¾ x 5½ in., illus., 15 + 455 + 6 pp. Manchester, England, The Scientific Publishing Company, 6 shillings net.

This volume is an attempt, according to the preface, to introduce the student to the main principles of alternating currents and alternating-current machinery from an experimental, rather than from an abstract theoretical, standpoint. The experiments are described in great detail, and the author hopes that the book may be of use to the young engineer engaged in handling or testing alternating-current machinery, as well as to the student in the laboratory. The present edition, the preface states, has been fundamentally revised, and, in order to make room for new matter, many sections of the earlier edition have been much condensed. Many additions have been made to the chapters on transformers and alternators, while the chapter on induction has been largely rewritten, and a chapter on single-phase motors added. The Contents are: Alternating Electromotive Force and Current; Impedance; Power and Power-Factor; Virtual Value of an Alternating Current; Effect of Capacity; The Transformer; Alternators; Synchronous Motors; The Polyphase Circuit; The Rotary Converter; The Polyphase Induction Motor; Single-Phase Motors; The Composition of Waves; Index.

AZIMUTH.

By George L. Hosmer. Leather, 7½ x 4¾ in., illus., 5 + 73 pp. New York, John Wiley & Sons; London, Chapman & Hall, Limited, 1909. $1.00 net. (Donated by the Author and the Publishers.)

The preface of this volume states the author's purpose to be to present, in compact form, certain approximate methods of determining the true bearing of a line, with the necessary rules and tables arranged in a simple manner, so that they will be useful to the practical surveyor. This is a handbook, it is stated, rather than a textbook, and for this reason many subjects, ordinarily included in books on practical astronomy, but not essential in learning to make the observations here described, have been omitted. The author has endeavored, he declares, to present the subject so that a person unfamiliar with astronomy will be able to apply these methods and obtain satisfactory results without taking time to master completely the underlying theory. The rules and tables have been put in compact form, so that the book may be carried into the field, and, while "the methods presented are not new, much of the valuable matter written on this subject is so scattered that it is difficult to find, in one small book, all that would be needed by the surveyor in making azimuth observations, and these are the reasons given by the author for presenting this volume."

THE NEW BUILDING ESTIMATOR.

A Practical Guide to Estimating the Cost of Labor and Material in Building Construction, from Excavation to Finish, With Various Practical Examples of Work Presented in Detail, and With Labor Figured Chiefly in Hours and Quantities: A Handbook for Architects, Builders, Contractors, Appraisers, Engineers, Superintendents, and Draftsmen. By William Arthur. Cloth, 6¾ x 4½ in., illus., 4 + 424 pp. New York, David Williams Company, 1909. $2.50 net.

This book is really a fifth edition, revised and enlarged, of "The Building Estimator," the first edition of which was published in 1904. In the preparation of the material for the work, the author states that he has drawn not only upon his own wide experience as an estimator and builder, but that he has obtained from scores of others, who were best qualified to furnish information, the data on quantities and costs pertaining to a great number of representative structures, large and small. The prices in this edition are said to be brought up to date, and much new tabulated matter has been added.

EIN NEUES SCHNELLBAHNSYSTEM: VORSCHLÄGE ZUR VERBESSERUNG DES PERSONEN-VERKEHRS.

Von August Scherl. Boards, 11¼ x 14 in., illus., 6 + 122 pp. Berlin, August Scherl, 1909.

The author of this work states that the ideas and propositions here laid before the public have been developed by many years of observation, and that by them he hopes to offer a remedy for a deep-seated evil. The reader will find, he states, that the modern transit problem is taken up from an entirely new side, by entirely

new technical methods, which, in their details, have been worked out by prominent specialists, tested by the first authorities, and pronounced to be the traffic methods of the future. The Contents are: Kapitel I, Die Krisis im Gegenwärtigen Eisenbahnwesen; Kapitel II, Das Neue System—Allgemeine Prinzipien des Personenverkehrs, Die Organisation, Die Technische Durchführung des Neuen Systems (Das Einschienensystem, Das Rollende Material, Die Strecke, Die Bahnhöfe), Der Betrieb; Kapitel III, Wirtschaftliche und Soziale Ausblicke.

EXPLOITATION DES PORTS MARITIMES.

Par De Cordemoy. Leather, 7½ x 5 in., 8 + 560 pp. Paris, H. Dunod et E. Pinat, 1909. 9 francs.

The Chapter headings are: Généralités; Régime des Ports en France; Ports Français et Etrangers; Les Ports Francs; Installations Générales-Hangars; Magasins et Quais; Manutention dans les Ports; Machinerie des Ports; Divers; Réparation des Navires—Cales; Réparation des Navires—Formes de Radoub; Réparation des Navires—Docks Flottants; Fermeture des Bassins de Radoub; Des Droits de Port; Développement de Quelques Ports; Règlements.

"HÜTTE": DES INGENIEURS TASCHENBUCH.

Herausgegeben vom Akademischen Verein Hütte. Zwanzigste Auflage. Leather, 7½ x 4½ in., illus., 3 v. Berlin, Wilhelm Ernst & Sohn, 1908-09. 20 marks.

The preface, which is signed by the Chairman of the Pocketbook Committee of the "Akademischen Verein Hütte," states that the first edition of the "Hütte" was issued in one volume, divided into three parts, each forming a complete whole. Later editions were issued in two volumes, but with the present edition, the compilers have returned to the original division. The first two of these volumes, which contain material for the mechanical engineer and the shipbuilder, were issued together in 1908, and the third, devoted to matters of interest to the construction engineer, has been issued separately this year. The preface states that much valuable matter has been added, and important chapters have been entirely rewritten; that the aim constantly before the Committee was to keep in close touch with the leading practice of the day and to select without prejudice only such data, both in text and illustration, as would be useful in actual practice. The Contents are: Abteilung I, Mathematik; Mechanik Starrer Körper; Mechanik Tropfbar Flüssiger Körper; Wärme; Festigkeitslehre; Stoffkunde; Eisenhüttenkunde; Maschinenteile; Anhang. Abteilung II, Kraftmaschinen; Arbeitsmaschinen; Schiffbau und Schiffsmaschinenbau; Eisenbahnbetriebsmittel; Elektrotechnik; Abteilung III, Vermessungskunde; Hochbau; Lüftung und Heizung; Strassenbau; Wasserversorgung; Städteentwässerung; Statik der Baukonstruktionen; Eisenbetonbau; Brückenbau; Eisenbahnbau; Drahtseilbahnen; Zahnradbahnen; Wasserbau; Gasfabrikation; Fabrikanlagen; Sachverzeichnis der I., II., und III. Abteilung.

BROWN'S DIRECTORY OF AMERICAN GAS COMPANIES: GAS STATISTICS, 1909.

Compiled and Corrected Annually by E. C. Brown. Cloth, 10 x 6½ in., 417 pp. New York, Press of the *Progressive Age*, 1909. $5.00.

This directory contains statistics of gas companies in the United States, Canada, South America, and Cuba, and of electrical companies where they are operated under the same name as the gas companies. Acetylene, gasolene and natural gas companies are included, also a list of parent or operating companies. There is an appendix containing brief historical data and financial status of the companies reported, a list of officers and members of gas associations in the United States, and a bibliography of books for gas men, divided by subject. The arrangement of the book is alphabetical.

EXERCISES IN SURVEYING FOR FIELD WORK AND OFFICE WORK.

With Questions for Discussion; Intended for Use in Connection with the Author's Book, "Plane Surveying." By John Clayton Tracy. Morocco, 7½ x 5 in., 14 + 169 pp. New York, John Wiley & Sons; London, Chapman & Hall, Limited, 1909. $1.00 net.

Part I of this book, which, as the title indicates, is to be used with the author's book, "Plane Surveying," consists of exercises in field work grouped according to the nature of the work, while Part II consists of exercises in office computations and plotting. The whole, the author asserts, constitutes a thorough introductory course in principles and methods. Each exercise begins with a refer-

ence to "Plane Surveying," and nearly all end with series of questions, which, the author states, should prove particularly valuable to young men who are seeking to perfect themselves in the knowledge of surveying. The Contents are: Part I, Field Work—Introductory Exercises; Exercises in Chaining; Exercises in the Use of the Transit; Exercises in Running Transit Lines; Transit Surveys; Special Problems in Transit Surveying; Exercises in Leveling; Special Problems in Leveling; Exercises in the Use of the Compass; Exercises in the Use of the Stadia; Exercises in the Use of the Plane Table; Exercises in Topographic Surveying; Exercises in Determining the True Meridian; A Study of Surveying Instruments; Adjustments; Part II, Office Work—General Methods of Computation; Calculation of Bearings; Latitudes and Departures; Triangulation; Omitted Measurements; Areas; Earthwork Calculations; Exercises in Plotting; Questions Pertaining to Mapping. There is also a detachable blank Field Note Book at the end of the book, of which duplicates are obtainable.

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A Short Hand-Book of Oil Analysis. By Augustus H. Gill. Fifth Edition, Revised and Enlarged. Philadelphia, J. B. Lippincott Co., 1909.

The Theory of Electric Cables and Networks. By Alexander Russell. New York, D. Van Nostrand Company, 1909.

The Beautifying of San Jose, A Report to the Outdoor Art League. By Charles Mulford Robinson. San Jose, 1909.

Versuche mit Exzentrisch Belasteten Betoneisernen Säulen. Von Dr. Maximilian Ritter von Thullie. Berlin, Wilhelm Ernst & Sohn, 1909.

Principles of Sewage Treatment. By Professor Dr. Dunbar. Translated by H. T. Calvert. London, Charles Griffin & Company, Limited, 1908.

British Rainfall 1907, On the Distribution of Rain in Space and Time over the British Isles during the Year 1907 as Recorded by More than 4 000 Observers in Great Britain and Ireland and Discussed with Articles upon Various Branches of Rainfall Work. By Hugh Robert Mill. Forty-seventh Annual Volume. London, Edward Stanford, 1908.

The First International Road Congress, Paris, 1908. By G. Montagu Harris and H. T. Wakelam. London, Wyman & Sons, Limited.

Thirty-Sixth Annual Report of the Local Government Board, 1906–07, Supplement Containing the Report of the Medical Officer for 1906–07. London, Wyman and Sons, Ltd.; Edinburgh, Oliver & Boyd; Dublin, E. Ponsonby, 1908.

An Historical Review of Waterways and Canal Construction in New York State. By Henry Wayland Hill. Buffalo, N. Y., Buffalo Historical Society, 1908.

Zentralblatt der Bauverwaltung, Herausgegeben im Ministerium der Öffentlichen Arbeiten. Jahrgänge 16–28, 1896–1908. Berlin, Wilhelm Ernst & Sohn, 1896–1908.

Gesundheits Ingenieur: Zeitschrift für die Gesamte Städtehygiene. Jahrgänge 26–31; 1903, '05–08. München und Berlin, R. Oldenbourg, 1903–08.

In Chancery of New Jersey, Mayor and Aldermen of Jersey City *vs.* Jersey City Water Supply Co. Vol. 9–10. Paterson, N. J., The Paterson Chronicle Co., 1909.

Chaudières Marines et Accessoires, avec Notes de Résistance des Matériaux et de Thermodynamique. Par L. Jauch et A. Masméjean. Two Volumes. Toulon, J. Alté; Paris, H. Dunod et E. Pinat, 1909.

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The Building Foreman's Pocket Book and Ready Reference. By H. G. Richey. New York, John Wiley & Sons; London, Chapman & Hall, Limited, 1909.

Tables of the Properties of Steam and other Vapors and Temperature-Entropy Table. By Cecil H. Peabody. Eighth Edition, Rewritten. New York, John Wiley & Sons; London, Chapman & Hall, Limited, 1909.

Practical Testing of Gas Meters. By C. H. Stone. New York, John Wiley & Sons; London, Chapman & Hall, Limited, 1909.

Railroad Promotion and Capitalization in the United States. By Frederick A. Cleveland and Fred Wilbur Powell. New York, Longmans Green and Co., 1909.

Theory and Calculation of Transient Electric Phenomena and Oscillations. By Charles Proteus Steinmetz. New York, McGraw Publishing Company, 1909.

Cours de Ponts Métalliques, Professé à l'École Nationale des Ponts et Chaussées. Par Jean Résal. Tome 1, Encyclopédie des Travaux Publics. Paris, Librairie Polytechnique, Ch. Beranger, Éditeur, 1908.

Handbuch der Ingenieurwissenschaften, Vol. 3, Part IV, Die Baumaschinen. Von F. Lincke. Zweite vermehrte Auflage. Leipzig, Wilhelm Engelmann, 1908.

Disposal of Liquid Refuse from Distilleries: Sixth Report of the Commissioners Appointed to Inquire and Report What Methods of Treating and Disposing of Sewage (Including any Liquid from any Factory or Manufacturing Process) May Properly Be Adopted. London, Wyman and Sons, Limited, 1909.

Principles of Mining, Valuation, Organization and Administration, Copper, Gold, Lead, Silver, Tin and Zinc. By Herbert C. Hoover. New York, Hill Publishing Company, 1909.

The Westinghouse E-T Air Brake Instruction Pocket Book; A Complete Work Explaining in Detail the Improved Westinghouse Locomotive Air-Brake Equipment, Including Both the No. 5 and the Latest Perfected No. 6 Style; Contains Examination Questions and Answers. By W. W. Wood. New York, The Norman W. Henley Publishing Co., 1909.

Electric Lamps. By Maurice Solomon. New York, D. Van Nostrand Company, 1908.

Heavy Electrical Engineering. By H. M. Hobart. New York, D. Van Nostrand Company, 1908.

Technisches Gemeindeblatt; Zeitschrift für die technischen und hygienischen Aufgaben der Verwaltung; Organ der Vereinigung der technischen Oberbeamten der deutschen Städte. Herausgegeben von H. Albrecht. Vols. 1–11. Berlin, Carl Heymans, 1899–1909.

Moody's Manual of Railroads and Corporation Securities, 1909. Tenth Annual Number. New York, Moody Manual Company, 1909.

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The Mineral Industry, Its Statistics, Technology and Trade during 1908. Edited by Walter Renton Ingalls. Vol. XVII. New York, Engineering and Mining Journal, 1909.

Conservation of Natural Resources: The Annals of the American Academy of Political and Social Science. Vol. XXXIII, No. 3, May, 1909. Philadelphia, American Academy of Political and Social Science, 1909.

SUMMARY OF ACCESSIONS

(From May 11th to July 31st, 1909)

Donations (including 67 duplicates)	795
By purchase	60
Total	855

MEMBERSHIP

ADDITIONS

(May 12th to July 31st, 1909)

MEMBERS		Date of Membership.
AURYANSEN, FREDERICK. Bridge Engr., Long Island R. R., 44 Union Hall St., Jamaica, N. Y.	Jun.	Oct. 5, 1897
	Assoc. M.	Dec. 3, 1902
	M.	April 6, 1909
BARD, GEORGE PARKER, Boonton, N. J.		April 6, 1909
BAUER, JACOB LOUIS. 130 Broad St., Elizabeth, N. J.	Assoc. M.	Feb. 5, 1896
	M.	June 1, 1909
BERRY, JOHN BENNINGTON. Chf. Engr., C., R. I. & P. Ry. Co., La Salle Station, Chicago, Ill.		May 4, 1909
BETTES, CHARLES ROBERT. Chf. Engr., Queens Co. Water Co., Far Rockaway, N. Y.	Jun.	Oct. 1, 1890
	Assoc. M.	June 7, 1893
	M.	June 1, 1909
BETTS, ROMEO THOMPSON. Asst. Engr., Dept. Docks and Ferries, Pier A, N. R., New York City	Assoc. M.	Oct. 3, 1900
	M.	Feb. 2, 1909
BOWEN, OSCAR SIDNEY. Res. Engr., G. N. Ry. Co., Spokane, Wash.		May 4, 1909
BREWER, BERTRAM. City Engr. and Supt. of Sewers, Waltham, Mass.		June 1, 1909
BROWNE, JAMES SIMPSON. Div. Engr., N. Y., N. H. & H. R. R., 409 Union Station, Providence, R. I.	Assoc. M.	Oct. 4, 1893
	M.	July 1, 1909
BURGESS, GEORGE HECKMAN. Chf. Engr., The Delaware & Hudson Co., Albany, N. Y.	Jun.	Feb. 1, 1898
	Assoc. M.	June 3, 1903
	M.	July 1, 1909
BURROWES, HARRY GILBERT. Div. Engr., Hudson Companies, 43 West 27th St., New York City	Jun.	Sept. 2, 1902
	Assoc. M.	Feb. 3, 1904
	M.	July 1, 1909
CALDERWOOD, ISAAC GLIDDEN. Massena, N. Y.		June 1, 1909
CALDWELL, WILLIAM HOWELL. P. O. Drawer 795, Tampa, Fla.		May 4, 1909
CORNELL, JOHN NELSON HAYWARD. 60 Wall St., New York City	Jun.	May 1, 1889
	M.	June 1, 1909
CREW, CHARLES CORWIN. 1408 Fourth Ave., Bay City, Mich.	Assoc. M.	April 1, 1903
	M.	June 1, 1909
DARROW, FRANK TENNEY. Engr., M. of W., C., B. & Q. Lines West, Lincoln, Nebr.	Assoc. M.	May 1, 1907
	M.	May 4, 1909
DAVIS, LEONARD HENRY. Chf. Engr., The Michigan Lake Superior Power Co., The Lake Superior Power Co., Sault Ste. Marie, Ont., Canada	Assoc. M.	Nov. 5, 1902
	M.	May 4, 1909
DIEHR, ALVAH BENJAMIN. U. S. Junior Engr., Box 1017, Memphis, Tenn.	Assoc. M.	Nov. 5, 1902
	M.	July 1, 1909

MEMBERS (*Continued*).

		Date of Membership.
EVANS, ROBERT ROGERS. 24 Arlington St., Haverhill, Mass.		July 1, 1909
FENKELL, GEORGE HARRISON. Civ. Engr. to Water Commrs. (Res., 334 Field Ave.), Detroit, Mich.	Jun.	Jan 3, 1899
	Assoc. M.	Oct. 1, 1902
	M.	July 1, 1909
FENN, WILLIAM HENRY. Room 226, DuPont Bldg., Wilmington, Del.		July 1, 1909
FERGUSON, JOHN BERTON. Hagerstown, Md.		July 1, 1909
FLETCHER, AUSTIN BRADSTREET. 15 Ashburton Pl., Boston, Mass		June 1, 1909
FRENCH, JAMES ADAMS. Care, U. S. Reclamation Service, Engle, N. Mex.		July 1, 1909
GETMAN, FRANK LAWTON. Engr. in Chg. of Office, Cuban Eng. & Contr. Co., 438 Lonja del Comercia, Havana, Cuba.	Assoc. M.	April 5, 1905
	M.	May 4, 1909
GOODWIN, GEORGE ESTYN. Project Engr., U. S. Reclamation Service, Family, Mont.	Assoc. M.	Jan. 8, 1908
	M.	July 1, 1909
GRACE, ARTHUR. Medicine Hat, Alta., Canada		July 1, 1909
HARDY, HARRY. Care, Panama R. R., Colon, Panama	Assoc. M.	June 1, 1892
	M.	June 1, 1909
HARKNESS, GEORGE EDWARD. 60 City Hall, Boston, Mass.		June 1, 1909
HASLAM, ERWIN ERNEST. Box 235, Tupper Lake, N. Y.		July 1, 1909
HAWGOOD, HARRY. Cons. Engr., 722 H. W. Hellman Bldg., Los Angeles, Cal.		May 4, 1909
HEWETT, WILLIAM SHERMAN. Box 1004, Minneapolis, Minn		June 1, 1909
HITTELL, JOHN BENJAMIN. Chf. Engr. of Streets, Board of Local Impvts., 200 Randolph St., Room 300, Chicago, Ill.		July 1, 1909
HOYT, JOHN CLAYTON. Asst. Chf. Hydrographer, U. S. Geological Survey, 1330 F St., N. W., Washington, D. C.	Assoc. M.	May 4, 1904
	M.	June 1, 1909
HYDE, CHARLES GILMAN. San. and Hydr. Engr.; Associate Prof., San Eng., Univ. of California, Berkeley, Cal.	Assoc. M.	April 2, 1902
	M.	June 1, 1909
JACOB, THOMAS NOTTINGHAM. Chf. Engr., The East Side Levee and San. Dist., 205 Sexton Bldg., East St. Louis, Ill.		May 4, 1909
JACOBS, JOSEPH. Care, Porto Rico Irrig. Service, Guayama, Porto Rico.	Assoc. M.	Sept. 4, 1901
	M.	April 6, 1909
JAMES, WILLIAM ATLEE. Box 85, Kenora, Ont., Canada		July 1, 1909
JOHNSON, ARTHUR NEWHALL. State Highway Engr., Springfield, Ill.	Assoc. M.	Sept. 2, 1903
	M.	July 1, 1909
JONES, HOWARD MURFREE. Room 48, Cole Bldg., Nashville, Tenn		June 1, 1909
LAKE, EDWARD NELSON. 321 Center St., Wheaton, Ill.		July 1, 1909

MEMBERS (*Continued*).

	Date of Membership.
LEONARD, JOHN BUCK. 813 Sheldon Bldg., San Francisco, Cal	June 1, 1909
MAIN, CHARLES THOMAS. 45 Milk St., Room 911, Boston, Mass	July 1, 1909
MANLEY, LAURENCE BRADFORD. 22 Garden St., West Roxbury, Mass	July 1, 1909
MARTIN, EDGAR DARWIN. 172 Washington St., Chicago, Ill	July 1, 1909
MASON, FRANCIS. Res. Engr., P. T. & T. R. R. Co., 345 East 33d St., New York City...	Jun. April 3, 1900 Assoc. M. April 1, 1903 M. July 1, 1909
MEREDITH, WYNN. 907 Union Trust Bldg., San Francisco, Cal	June 1, 1909
MITCHELL, WILLIAM SELBY. U. S. Engr. Office, Custom House, St. Louis, Mo	June 1, 1909
MONROE, RICHARD. U. S. Asst. Engr., U. S. Engr. Office, Rock Island, Ill	June 1, 1909
O'BRIEN, ARTHUR. City Engr., Utica, N. Y	July 1, 1909
PAGE, LOGAN WALLER. Director, Office of Public Roads, U. S. Dept. of Agriculture, 2223 Mass. Ave., Washington, D. C	July 1, 1909
PARKER, FREDERICK YANCY. U. S. Engr. Office, Custom House, St. Louis, Mo	June 1, 1909
PATTERSON, WILLIAM RODNEY. 1448 Monadnock Blk., Chicago, Ill	May 4, 1909
PERRY, CHAUNCY RUSCH. Diamond Pond, *via* Colebrook, N. H	Assoc. M. Feb. 1, 1905 M. June 1, 1909
PEYTON, JOHN HOWE. Cons. Engr., North Coast R. R. Co., Spokane, Wash	June 1, 1909
PHILLIPS, WILLIAM RENTON. 419 Lumber Exchange Bldg., Portland, Ore	June 1, 1909
PITCHER, SAMUEL HENRY. 418 Main St., Worcester, Mass	Mar. 2, 1909
RAIKES, HUGH PERCIVAL. 63 Temple Row, Birmingham (also 15 Victoria St., Westminster, London), England	Assoc. M. June 6, 1906 M. July 1, 1909
ROHRER, GRANT. 299 Broadway, New York City	July 1, 1909
ROURKE, LOUIS KEEGAN. Asst. Div. Engr., Central Div., Isthmian Canal Comm., Empire, Canal Zone, Panama	July 1, 1909
SAYLES, EARLE WILLOUGHBY. City Engr., City Hall, Watertown, N. Y	June 1, 1909
SCHROEDER, CHRISTOFFER. 6509 LaFayette Ave., Chicago, Ill	June 1, 1909
SELLEW, FRANCIS LIBBY. Project Engr., U. S. Reclamation Service, Yuma, Ariz	Assoc. M. Dec. 5, 1906 M. July 1, 1909
THOMAS, DAVID GORTON. Chf. Engr., Denver Union Water Co., Denver, Colo	June 1, 1909

MEMBERS (*Continued*).

	Date of Membership.
TORRANCE, WILLIAM MARTIN. Constr. Engr., Merrill-Ruckgaber Co., 50 Church St., New York City	July 1, 1909
TOWER, MORTON LOUDON. U. S. Engr. Office, 665 Monadnock Bldg., San Francisco, Cal.	Assoc. M. May 6, 1908 M. July 1, 1909
VAN NORDEN, RUDOLPH WARNER. Cons. Engr., 912 Union Trust Bldg., San Francisco, Cal	July 1, 1909
VINCENT, EDWIN DERICKSON. Res. Engr., U. S. Reclamation Service, Laguna Dam, *via* Yuma, Ariz	May 4, 1909
WALDRON, SAMUEL PAYSON. Care, Am. Bridge Co., 30 Church St., New York City	Assoc. M. Jan. 7, 1903 M. May 4, 1909
WILLIAMS, FRIEND PITTS. Res. Engr., N. Y. State Barge Canal, 352 Guy Park Ave., Amsterdam, N. Y	July 1, 1909
WILLOUGHBY, JULIUS EDGAR. Engr. of Constr., L. & N. R. R. Co., 9th and Broadway, Louisville, Ky	June 1, 1909
WINDSOR, PHILIP BRUNDAGE. 1 Madison Ave., New York City	July 1, 1909

ASSOCIATE MEMBERS

ACKENHEIL, ALFRED CURTIS. Box 53, Rockville Center, N. Y.	April 6, 1909
AREND, ALBERT CORNELIUS. Cons. Engr., Room 691, Brandeis Bldg., Omaha, Nebr	Feb. 2, 1909
BARKER, CHARLES WHITNEY TILLINGHAST. Pres., Field, Barker & Underwood, Inc., Engrs. and Contrs., 718 Arcade Bldg., Philadelphia, Pa.	Jun. June 6, 1905 Assoc. M. June 1, 1909
BEACH, WILLIAM NICHOLAS. 26 Cortlandt St., New York City	July 1, 1909
BURNS, LOUIS ANDREW. Barge Canal Office, Rome, N. Y.	July 1, 1909
BURTON, WILLIAM. Res. Engr., Detroit River Tunnel Co., Windsor, Ont., Canada (Res., 108 Charlotte Ave., Detroit, Mich.)	Jun. Oct. 3, 1905 Assoc. M. May 4, 1909
CAMPBELL, DUNCAN HUGH. Care, Montana R. R. Co., Dorsey, Mont	July 1, 1909
CAREY, EDWARD GILMAN. 688 Dawson St., New York City.	July 1, 1909
CHARNLEY, WALTER. Caixa 58, Rio Grande, Rio Grande do Sul, Brazil	May 4, 1909
CLAPP, FRANK LEMUEL. Cornwall-on-Hudson, N. Y.	May 4, 1909
CONWAY, JOHN SEBASTIAN. Asst. Superv. Engr. U. S. Reclamation Service, North Yakima, Wash	May 4, 1909
COVERT, CLERMONT C. Dist. Engr., U. S. Geological Survey, Room 18, Federal Bldg., Albany, N. Y.	July 1, 1909
DAVIS, JOHN CHARLES. Dean of Coll. of Applied Science and Eng., Marquette Univ., Milwaukee, Wis	July 1, 1909
EDELEN, THOMAS JEFFERSON STONE. 187 Garry St., Winnipeg, Man., Canada	July 1, 1909

ASSOCIATE MEMBERS (*Continued*).

		Date of Membership.
ELLIOTT, MALCOLM. Prin. Hydrographer, Office of Chf. Engr., Isthmian Canal Comm., Culebra, Canal Zone, Panama	Jun.	Nov. 5, 1907
	Assoc. M.	April 6, 1909
ETCHEVERRY, BERNARD ALFRED. Agriculture Bldg., Univ. of California, Berkeley, Cal		June 1, 1909
FOUNTAIN, THOMAS LILLY. Civ., San., Elec. and Mech. Engr. (The Fountain-Shaw Eng. Co.), Binz Bldg., Houston, Tex	Jun.	April 2, 1907
	Assoc. M.	April 6, 1909
FRYE, HARLEY EDGAR. U. S. Engr. Office, Zanesville, Ohio		June 1, 1909
GERSBACH, EDWARD CHARLES. Care, U. S. Reclamation Service, Browning, Mont		June 1, 1909
GIBBS, ELBERT ALLAN. Asst. Engr., McClintic-Marshall Constr. Co., Pittsburg, Pa	Jun.	Mar. 6, 1906
	Assoc. M.	July 1, 1909
GILBERT, ARCHIBALD MARVINE. Asst. Engr., Twin Falls Salmon River Land & Water Co., Twin Falls, Idaho		May 4, 1909
GILLEN, WALTER JOSEPH. Asst. Engr., Board of Water Supply, New York City, Brown Station, Ulster Co., N. Y.	Jun.	Oct. 31, 1905
	Assoc. M.	July 1, 1909
GRAM, LEWIS MERRITT. 1047 Spitzer Bldg., Toledo, Ohio		June 1, 1909
GRAY, HENRY LILBURN. Engr. of R. R. Comm. of Wash., Olympia, Wash		July 1, 1909
HARDESTY, JAMES ROBERT. Care, Virginia Bridge & Iron Co., Roanoke, Va		Feb. 2, 1909
HARTMAN, RUSSELL THEODORE. Care, Des Moines Bridge & Iron Works, Des Moines, Iowa		May 4, 1909
HATTON, HERBERT WATSON. Care, T. Chalkley Hatton, 302 Equitable Bldg., Wilmington, Del	Jun.	April 4, 1905
	Assoc. M.	June 1, 1909
HAZELTON, WILLIAM SYLVESTER. 145 W. Euclid Ave., Detroit, Mich		May 4, 1909
HECK, NICHOLAS HUNTER. U. S. Coast Survey, Washington, D. C		July 1, 1909
HILL, GEORGE SAMUEL. Cons. Engr. (Lathrop & Hill), 1305 Call Bldg., San Francisco, Cal	Jun.	Jan. 3, 1907
	Assoc. M.	June 1, 1909
HILTON, HARRY LEGRAND. Box 604, Cornwall-on-Hudson N. Y.		June 1, 1909
HUFF, CLYDE LESLIE. Care, The Arnold Co., Moore, Idaho		July 1, 1909
HUNTER, THOMAS BENTON. Res. Engr., Monterey County Water Works, Monterey, Cal	Jun.	April 4, 1905
	Assoc. M.	July 1, 1909
JORGENSEN, LARS RASMUS. 1406 Chronicle Bldg., San Francisco, Cal		May 4, 1909
KELLEY, MATTHEW DETOBIN. Licensed Civ. Engr. and Surv., Lander, Wyo	Jun.	Sept. 3, 1907
	Assoc. M.	June 1, 1909

ASSOCIATE MEMBERS (*Continued*).

	Date of Membership.
KING, CLIFFORD MARSHALL. City Engr., Sandusky, Ohio ... Jun.	Mar. 6, 1906
... Assoc. M.	July 1, 1909
KISSACK, ALFRED BROUGHTON. 903 S. Third St., Leavenworth, Kans	May 4, 1909
LAFLER, WILLIAM ARTHUR. P. O. Box 351, Whitesboro, N. Y	May 4, 1909
LAMONT, CLARENCE BOOTH. 1009 Cherry St., Seattle, Wash.	May 4, 1909
LAURGAARD, OLAF. Asst. Engr., U. S. Reclamation Service, Conconully, Wash	June 1, 1909
LINEBERGER, WALTER FRANKLIN. Apartado 333, Torreón, Coah., Mexico	May 4, 1909
LONGLEY, FRANCIS FIELDING. Care, City Engr., Toronto, Ont., Canada ... Jun.	Feb. 28, 1905
... Assoc. M.	Mar. 2, 1909
MCBEE, VARDRY ECHOLS, JR. Care, James Team, Lugoff, S. C	Oct. 7, 1908
MARTIN, BERTRAND CLIFFORD. Asst. Dist. Engr., N. Y. C. & H. R. R. R. Co., 163 Flower Ave., East, Watertown, N. Y	May 4, 1909
MATLAW, ISAAC SOLON. Asst. Engr., Dept. of State Engr. and Surv., Vischer Ferry, N. Y. ... Jun.	Oct. 2, 1906
... Assoc. M.	July 1, 1909
MEIER, WILLIAM. 81 Alice Pl., Chicago, Ill	June 1, 1909
MOSS, ROBERT FAULKNER. Columbia, Mo ... Jun.	Nov. 1, 1904
... Assoc. M.	Mar. 2, 1909
NASH, FRANK DANA. Div. Engr., Bureau of Public Works, Manila, Philippine Islands	May 4, 1909
PADDOCK, HOWARD CHARLES. 1916 Eighty-fifth St., Brooklyn, N. Y	June 1, 1909
PATTERSON, CLAIR BRANDON. Res. Engr., L. S. & M. S. Ry., Darrowville, Ohio	July 1, 1909
PELLISSIER, GEORGE EDWARD. Chf. Engr., Goldschmidt Thermit Co., 90 West St., New York City ... Jun.	Feb. 28, 1905
... Assoc. M.	July 1, 1909
PETTEBONE, LAUREN AUGUSTUS. 307 Buffalo Ave., Niagara Falls, N. Y	Feb. 2, 1909
PHILLIPS, WILLIAM HALE. Box 1023, Spokane, Wash ... Jun.	Nov. 1, 1904
... Assoc. M.	May 4, 1909
POND, HARRY BRADFORD. 1432 Crotona Park, East, New York City	May 4, 1909
POOLE, JOHN HUDSON. Capt., Corps of. Engrs., U. S. A., Room 150, War Dept., Washington, D. C	June 1, 1909
PRATT, JOEL MARSH. 24 South Catherine St., Mobile, Ala.	July 1, 1909
RHOADES, THEODORE ECKFORD. Chf. Engr., Whitney-Steen Co., 135 Broadway, New York City	June 1, 1909
RICE, RAY HOWARD. The Monitor, Granby St., Norfolk, Va.	July 1, 1909

ASSOCIATE MEMBERS (*Continued*).

	Date of Membership.
SHELEY, HORACE WEST. Civ. and Min. Engr., P. O. Box 34, Salt Lake City, Utah..... Jun.	Sept. 5, 1905
Assoc. M.	July 1, 1909
SMITH, WILLIAM ERNEST. Care, Eng. Constr. Co., Washington, Kans.....	Dec. 1, 1908
SWARTWOUT, ROY ADOLF. Chf. Engr., Western Bridge & Constr. Co., 340 Bee Bldg., Omaha, Nebr..... Jun.	May 2, 1905
Assoc. M.	July 1, 1909
SWEENEY, HARRY CLINTON. 790 President St., Brooklyn, N. Y.....	July 1, 1909
THOMAS, WILLIAM EDWARD. Colonial Hall, Jamaica, N. Y.	July 1, 1909
TILT, GARRET EDWARD. Milliken, N. Y.....	June 1, 1909
TUDBURY, WARREN CHAMBERLAIN. Asst. Engr., N. Y. C. & H. R. R. R. Co., 145 Washington St., Buffalo, N. Y..	June 1, 1909
WALDRON, ALBERT EDWIN. Capt., Corps of Engrs., U. S. A., U. S. Engr. Office, P. O. Box 1092, Dallas, Tex.....	July 1, 1909
WILLARD, NORMAN RAND. Camp La Prele, Inez, Wyo.....	June 1, 1909
WILLIAMS, JACOB PAUL JONES. Asst. Engr., Designer, McClintic-Marshall Constr. Co., 10 Trenton Ave., Swissvale, Pa.....	July 1, 1909
WINTERHALTER, LEO P. Asst. Engr. on Constr., Osborn Eng. Co., Cleveland, Ohio.....	July 1, 1909
YAPPEN, ADOLPH. 430 North Francisco Ave., Chicago, Ill..	June 1, 1909

ASSOCIATE

HAYS, JOHN COFFEE. Visalia, Cal.....	May 4, 1909

JUNIORS

ASHBROOK, CHESTER DANIEL. Care, King County Engr.'s Office, Seattle, Wash.....	May 4, 1909
BERGMAN, HARRY MONTIFIORE. 2422 Morris Ave., Bronx, New York City.....	May 4, 1909
BESWICK, JAMES EVERETT. Cold Spring, Putnam Co., N. Y.	July 1, 1909
BISHOP, LYMAN EDGAR. 3787 Franklin, Denver, Colo.....	May 4, 1909
BROOKS, JOSIAH RICHARDSON. 603 N. Main St., Mt. Vernon, Ohio	Feb. 2, 1909
BURR, MYRON CARLOS. (Burr & Ferguson), 216 Loo Bldg., Vancouver, B. C., Canada.....	May 4, 1909
CHAPMAN, ARNOLD GOODWIN. 52 Chestnut St., Albany, N. Y.....	July 1, 1909
CUTLER, STANLEY GARDNER. Asst. Engr., Bureau of Public Works, Manila, Philippine Islands.....	May 4, 1909
HOLLAND, HOWARD KINGSBURY. 207 New Eng. Bldg., Ann Arbor, Mich.....	June 1, 1909
HORRIGAN, WILLIAM JAMES. 405 Ford Bldg., Wilmington, Del.....	June 1, 1909
HOWE, FRANK RAY. 66 Jamaica Ave., Flushing, N. Y....	June 1, 1909

JUNIORS (*Continued*).

	Date of Membership.
HOWES, FRANKLIN JOHNSON. 34 Clinton Ave., Rochester, N. Y.	June 1, 1909
JONES, BENJAMIN EARL. Care, Office of Public Roads, Washington, D. C.	April 6, 1909
KRIEGER, ALBERT AUGUST. 1141 Cherokee Rd., Louisville, Ky	June 1, 1909
LARIMER, ROBERT SHERMAN. Estes Park, Colo.	June 1, 1909
LEMCKE, KARL WOLFGANG. Box 383, Ambridge, Pa.	June 1, 1909
MYERS, GEORGE HIGGINS. 219 McPhee Bldg., Denver, Colo.	June 1, 1909
NICHOLS, JOHN ROBERT. Hotel Buckminster, Boston, Mass.	June 1, 1909
NORDWELL, ALFRED WORCESTER. 20 Shepard St., Cambridge, Mass	June 1, 1909
RANDOLPH, ROBERT ISHAM. 164 Dearborn St., Room 748, Chicago, Ill.	July 1, 1909
ROLLINS, ANDREW PEACH. Eagle Lake, Tex.	May 4, 1909
SIMS, HARVEY HILLYER. 154 Gordon St., Atlanta, Ga.	May 4, 1909
SMALL, GILBERT. 428 Lexington St., Waltham, Mass.	June 1, 1909
SMITH, JOSEPH. Care, Board of Water Supply, Brown Station, N. Y.	Feb. 2, 1909
SUAREZ Y CORDOVES, PATRICIO ANDRES. Gen. Mgr., Compañia Eléctrica de Alumbrado y Traccion, Apartado 110, Santiago de Cuba, Cuba.	April 6, 1909
TEICHERT, ADOLPH, JR. Gen. Contr., 2401 J St., Sacramento, Cal.	July 1, 1909
TILLSON, EDWIN DELEVAN. 179 Marcy Ave., Brooklyn, N. Y.	Mar. 2, 1909
WALTON, HARRY COLLINS. 1103 Park Row Bldg., New York City.	Jan. 5, 1909
WILLIAMS, CLEMENT CLARENCE. 1133 Pleasant St., Boulder, Colo	June 1, 1909
WINTERMUTE, FERD. CLARK. 414 Second National Bank, Wilkes-Barre, Pa.	July 1, 1909
WRIGHT, THOMAS JUDSON, JR. Churchland, Va.	June 1, 1909

CHANGES OF ADDRESS

MEMBERS

ALLEN, KENNETH. Engr., Met. Sewerage Comm. of New York, 17 Battery Pl., New York City.

ALLEN, WALTER HINDS. Civ. Engr., U. S. N., U. S. Naval Magazine, Hingham, Mass.

ARGOLLO, MIGUEL DE TEIVE E. Pres., Bahia General Transportation Co., Caixa Postal 89, Bahia, Brazil.

ARMSTRONG, WALTER ROOT. Asst. Supt., Idaho Div., Ore. Short Line R. R., Nampa, Idaho.

MEMBERS (*Continued*).

ASHMEAD, FRANK MILLIGAN. Asst. to Prin. Asst. Engr., B. & A. V. Div., P. R. R., 428 Richmond Ave., Buffalo, N. Y.

BACHERT, AUGUSTUS ELLSWORTH. Gen Supt., Rockhill Iron & Coal Co., Chf. Engr., East Broad Top R. R. Co., Tyrone, Blair Co., Pa.

BACOT, WILLIAM SINCLAIR. 21 Summit Pl., Utica, N. Y.

BAKENHUS, REUBEN EDWIN. Civ. Engr., U. S. N., U. S. Naval Station, Olongapo, Philippine Islands.

BALDWIN, ERNEST HOWARD. Const. Engr., U. S. Reclamation Service, Ellensburg, Wash.

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POWELL, WILLIAM JENNER. P. O. Box 1, Merkel, Tex.

QUERBACH, EARL. 3425 Wyandotte St., Kansas City, Mo.

REED, RALPH JOHN. 1012 Security Bldg., Los Angeles (Res., Whittier), Cal.

ROBBINS, HALLET RICE. Managing Engr., Colonial Gold Min. Co., Keremeos, B. C., Canada.

ROMMEL, WILLIAM GUSTAVE. Res. Engr., Bayless Pulp & Paper Co., Goodyear Hotel, Austin, Pa.

SAXE, VAN RENSSELAER POWELL. Asst. Engr. for Standard Concrete Steel Co., 415 East 31st St., New York City (Res., Hicksville, N. Y.).

SMITH, ELROY GEORGE. 335 West 58th St., New York City.

SMITH, FRANCIS MARSHALL. Asst. Engr., N. P. Ry., Room 27, Second and King Sts., Seattle, Wash.

STIRLING, VINCENT REYNOLDS. Care, Prof. S. G. George, Cornell Univ., Ithaca, N. Y.

STRATTON, CHARLES HEYWOOD. 410 W. Court St., Paris, Ill.

STROUT, GALE STANLEY. Room 902, Union Savings Bank Bldg., Oakland, Cal.

TALLMAN, PAUL BERTRAM. 46 Chestnut St., East Orange, N. J.

TRUE, ALBERT OTIS. Care, Hering & Fuller, 170 Broadway, New York City.

WADDELL, NEEDHAM EVERETT. McClintic-Marshall Constr. Co., Rankine, Pa.

WATKINS, GUY ANDERSON. 223 Southern Trust Bldg., Little Rock, Ark.

WATSON, GEORGE LINTON. Box 448, Dover, Del.

WEIDNER, CARL ROBERT. 18 W. Gilman St., Madison, Wis.

WESTOVER, HENRY CHRISTOPHER. 534 Beals Bldg., Kansas City, Mo.

WILEY, HUGH LEMUEL. Garberville, Cal.

WOLFF, MAX WILLIAM. U. S. Reclamation Service, Boise, Idaho.

WOOD, ROBERT WALTER. Asst. Engr. in Chg. of Port Richmond Div., Borough of Richmond, N. Y. City, 800 Post Ave., West New Brighton, N. Y.

YEO, WILLIAM ALBERT. 130 Bulls Ferry Rd., Weehawken Heights, N. J.

RESIGNATION

ASSOCIATE MEMBER	Date of Resignation.
HAMLIN, WILLIAM ELIOT	June 1, 1909

DEATHS

BALDWIN, HENRY FURLONG. Elected Associate Member, December 7th, 1892; Member, February 6th, 1895; died June 17th, 1909.

BUCK, LEFFERT LEFFERTS. Elected Member, February 3d, 1875; died July 17th, 1909.

DAVIS, FRANK LESLIE. Elected Member, September 6th, 1905; died June 9th, 1909.

DIMMICK, JOHN BAGLEY. Elected Member, July 10th, 1907; died June, 1909.

DUSENBERRY, WALTER LORTON. Elected Junior, November 5th, 1890; died May 13th, 1909.

FRENCH, GEORGE HARRISON. Elected Associate Member, December 5th, 1906; died July 9th, 1909.

GATCHELL, GEORGE SAMUEL. Elected Member, May 7th, 1884; died June 22d, 1909.

KERNOT, WILLIAM CHARLES. Elected Member, March 6th, 1889; died March 14th, 1909.

STEELE, HENRY MAYNADIER. Elected Associate Member, April 5th, 1893; Member, December 6th, 1899; died July 5th, 1909.

MONTHLY LIST OF RECENT ENGINEERING ARTICLES OF INTEREST

(May 11th to July 31st, 1909)

NOTE.—*This list is published for the purpose of placing before the members of the Society, the titles of current engineering articles, which can be referred to in any available engineering library, or can be procured by addressing the publication directly, the address and price being given wherever possible.*

LIST OF PUBLICATIONS

In the subjoined list of articles, references are given by the number prefixed to each journal in this list.

(1) *Journal,* Assoc. Eng. Soc., 31 Milk St., Boston, Mass., 30c.
(2) *Proceedings,* Engrs. Club of Phila., 1317 Spruce St., Philadelphia, Pa.
(3) *Journal,* Franklin Inst., Philadelphia, Pa., 50c.
(4) *Journal,* Western Soc. of Engrs., Monadnock Blk., Chicago, Ill.
(5) *Transactions,* Can. Soc. C. E., Montreal, Que., Canada.
(6) *School of Mines Quarterly,* Columbia Univ., New York City, 50c.
(7) *Technology Quarterly,* Mass. Inst. Tech., Boston, Mass., 75c.
(8) *Stevens Institute Indicator,* Stevens Inst., Hoboken, N. J., 50c.
(9) *Engineering Magazine,* New York City, 25c.
(10) *Cassier's Magazine,* New York City, 25c.
(11) *Engineering* (London), W. H. Wiley, New York City, 25c.
(12) *The Engineer* (London); International News Co., New York City, 35c.
(13) *Engineering News,* New York City, 15c.
(14) *The Engineering Record,* New York City, 12c.
(15) *Railroad Age Gazette,* New York City, 15c.
(16) *Engineering and Mining Journal,* New York City, 15c.
(17) *Electric Railway Journal,* New York City, 10c.
(18) *Railway and Engineering Review,* Chicago, Ill., 10c.
(19) *Scientific American Supplement,* New York City, 10c.
(20) *Iron Age,* New York City, 10c.
(21) *Railway Engineer,* London, England. 25c.
(22) *Iron and Coal Trades Review,* London, England. 25c.
(23) *Bulletin,* American Iron and Steel Assoc., Philadelphia, Pa.
(24) *American Gas Light Journal,* New York City, 10c.
(25) *American Engineer,* New York City, 20c.
(26) *Electrical Review,* London, England.
(27) *Electrical World,* New York City, 10c.
(28) *Journal,* New England Water-Works Assoc., Boston, Mass., $1.
(29) *Journal,* Royal Society of Arts, London, England, 15c.
(30) *Annales des Travaux Publics de Belgique,* Brussels, Belgium.
(31) *Annales* de l'Assoc. des Ing. Sortis des Ecoles Spéciales de Gand, Brussels, Belgium.
(32) *Mémoires et Compte Rendu des Travaux,* Soc. Ing. Civ. de France, Paris, France.
(33) *Le Génie Civil,* Paris, France.
(34) *Portefeuille Economiques des Machines,* Paris, France.
(35) *Nouvelles Annales de la Construction,* Paris, France.
(37) *Revue de Mécanique,* Paris, France.
(38) *Revue Générale des Chemins de Fer et des Tramways,* Paris, France.
(41) *Modern Machinery,* Chicago, Ill., 10c.
(42) *Proceedings,* Am. Inst. Elec. Engrs., New York City, 50c.
(43) *Annales des Pontes et Chaussées,* Paris, France.
(44) *Journal,* Military Service Institution, Governors Island, New York Harbor, 50c.
(45) *Mines and Minerals,* Scranton, Pa., 20c.
(46) *Scientific American,* New York City, 8c.
(47) *Mechanical Engineer,* Manchester, England.
(48) *Zeitschrift,* Verein Deutscher Ingenieure, Berlin, Germany.
(49) *Zeitschrift für Bauwesen,* Berlin, Germany.
(50) *Stahl und Eisen,* Düsseldorf, Germany.
(51) *Deutsche Bauzeitung,* Berlin, Germany.
(52) *Rigasche Industrie-Zeitung,* Riga, Russia.
(53) *Zeitschrift,* Oesterreichischer Ingenieur und Architekten Verein, Vienna, Austria.
(54) *Transactions,* Am. Soc. C. E., New York City, $5.
(55) *Transactions,* Am. Soc. M. E., New York City, $10.
(56) *Transactions,* Am. Inst. Min Engrs., New York City, $5.

(57) *Colliery Guardian,* London, England.
(58) *Proceedings,* Eng. Soc. W. Pa., 803 Fulton Bldg., Pittsburg, Pa., 50c.
(59) *Transactions,* Mining Inst. of Scotland, London and Newcastle-upon-Tyne, England.
(60) *Municipal Engineering,* Indianapolis, Ind., 25c.
(61) *Proceedings,* Western Railway Club, 225 Dearborn St., Chicago, Ill., 25c.
(62) *Industrial World,* 59 Ninth St., Pittsburg, Pa.
(63) *Minutes of Proceedings,* Inst. C. E., London, England.
(64) *Power,* New York City, 20c.
(65) *Official Proceedings,* New York Railroad Club, Brooklyn, N. Y., 15c.
(66) *Journal of Gas Lighting,* London, England, 15c.
(67) *Cement and Engineering News,* Chicago, Ill., 25c.
(68) *Mining Journal,* London, England.
(70) *Engineering Review,* New York City, 10c.
(71) *Journal,* Iron and Steel Inst., London, England.
(73) *Electrician,* London, England, 18c.
(74) *Transactions,* Inst. of Min. and Metal., London, England.
(75) *Proceedings,* Inst. of Mech. Engrs., London, England.
(76) *Brick,* Chicago, Ill., 10c.
(77) *Journal,* Inst. Elec. Engrs., London, England.
(78) *Beton und Eisen,* Vienna, Austria.
(79) *Forscherarbeiten,* Vienna, Austria.
(80) *Tonindustrie Zeitung,* Berlin, Germany.
(81) *Zeitschrift für Architektur und Ingenieurwesen,* Wiesbaden, Germany.
(82) *Dinglers Polytechnisches Journal,* Berlin, Germany.
(83) *Progressive Age,* New York City, 15c.
(84) *Le Ciment,* Paris, France.
(85) *Proceedings,* Am. Ry. Eng. and M. of W. Assoc., Chicago, Ill.
(86) *Engineering-Contracting,* Chicago, Ill., 10c.
(87) *Roadmaster and Foreman,* Chicago, Ill., 10c.
(88) *Bulletin* of the International Ry. Congress Assoc., Brussels, Belgium.
(89) *Proceedings,* Am. Soc. for Testing Materials, Philadelphia, Pa.
(90) *Transactions,* Inst. of Naval Archts., London, England.
(91) *Transactions,* Soc. Naval Archts. and Marine Engrs., New York City.
(92) *Bulletin,* Soc. d'Encouragement pour l'Industrie Nationale, Paris, France.
(93) *Revue de Métallurgie,* Paris, France, 4 fr. 50.
(94) *The Boiler Maker,* New York City, 10c.
(95) *International Marine Engineering,* New York City, 20c.
(96) *Canadian Engineer,* Toronto, Canada, 15c.
(97) *Turbine,* Berlin, Germany, 1 Mark.

LIST OF ARTICLES

Bridge.

On Preservation Work at the Old Bridge of Ayr.* Arthur Hamilton Wilson, Assoc. M. Inst. C. E. (63) Vol. 175.
The Stresses in a Suspension Bridge.* L. H. Chase. (12) Serial beginning May 7.
The Collapse of the Reinforced-Concrete Arch Bridge across the Illinois River at Peoria, Ill. (13) May 13.
A Neglected Splice for Timber Tension Members. Robert Fletcher, Assoc. Am. Soc. C. E. (13) May 20.
The 42d St. Bridge, Philadelphia, Pa.; an Arch with Steel Ribs Filled Between with Concrete.* (13) May 20; (14) May 22.
Reinforced-Concrete Trestles with Pile Bents and Girder-Slab Spans: Chicago, Burlington & Quincy Ry.* (13) May 20.
The Stability of Arches.* Henry Adams, M. Inst. C. E. (Paper read before the Civ. and Mech. Engrs.' Soc.) (96) May 28.
Computation of a Reinforced Concrete Arch Rib with Fixed Ends for a Proposed 12th Street Bridge at Oakland, California. A. W. Miller. (86) June 2.
The Pegu-Moulmein Railway and the Sittang River.* A Stewart Buckle. (12) Serial beginning June 4.
The New Bridge over the Wear.* (12) Serial beginning June 4.
Pile-and-Cylinder Foundations of Norfolk & Western Bridge No. 5, Elizabeth River, at Norfolk.* J. E. Crawford, M. Am. Soc. C. E. (Abstract of paper read before the Engrs. and Constrs.' Soc. of Tidewater, Va.) (13) June 10.
The Galveston Causeway.* F. E. Lister. (15) June 11.
Method of Constructing Concrete Abutments on Concrete Cylinders for a Skew Bridge across the North Shore Channel of the Sanitary District of Chicago.* (86) June 16.
The Pennsylvania Steel Company's Report on the Blackwell's Island Bridge.* C. R. Young. (96) June 25.
A Reinforced Concrete Arch Cement Block Face.* R. McDowall. (96) June 25.
Difficult Bridge Work in Great Britain.* (13) July.

*Illustrated.

Bridge—(Continued).

Erection of the New River Bridge at Glen Lyn, Va., by the Cantilever Method.* L. L. Jewel, Assoc. M. Am. Soc. C. E. (13) July 8.
Short-Span Bridges over City Crossings.* (14) Serial beginning July 10.
Concrete Bridge Specifications. T. H. Macdonald. (Paper read before the Iowa Cement Users' Assoc.) (96) July 16.
The Gualeguay River Bridge.* J. R. Garrod and P. J. Risden. (11) July 23.
Progress on the Manhattan Bridge.* (14) July 31.
Note sur le Pont d'Arciat.* Parent. (43) Mar.
Pont de Chemin de Fer sur le Song-Ma (Tonkin).* Ch. Dantin. (33) May 8.
Achèvement du Viaduc des Fades sur la Sioule (Puy-de-Dôme).* A. Dumas. (33) May 29.
Le Pont de Commerce à Arcs Conjugués sur la Meuse à Liège.* Th. Seyrig. (30) June.
Graphische Behandlung von Bogenträgern über Mehreren Oeffnungen und mit (in Wagerechter Richtung) Elastisch Nachgiebigen Unterstützungen.* A. Ostenfeld. (81) Vol. 4, 1909.
Nickelstahl für Eisenbrücken.* (Discussion of article in *Proceedings*, Am. Soc. C. E.) (50) May 19.
Die Kaiser Wilhelm-Brücke in Wilhelmshafen.* Karl Bernhard. (48) Serial beginning May 22.
Eisenbahnüberführung in Eisenbeton auf Bahnhof Lörrach der Grossh. badischen Staatseisenbahnen.* B. Kirsch. (78) Serial beginning July 2.

Electrical.

Force or Press Fits for Armature or Fly-Wheel Hubs.* Kenneth Moodie. (5) Vol. 22, Pt. 2.
Arc Welding.* C. B. Avel. (58) May.
The Application of the No-Load and Short-Circuit Diagram to the Design and Testing of Induction Motors.* Stanley P. Smith. (73) Serial beginning May 7.
The Failure of a Portion of a Steel Tower Electric Transmission Line in California.* Edwin Duryea, Jr., M. Am. Soc. C. E. (13) May 13.
The Hydro-Electric Power Plant of the Rio de Janeiro Tramway, Light & Power Company.* L. L. Perry. (27) May 13.
Interconnections of Electrical Equipment of Indiana Steel Company, Gary, Ind.* E. A. Lof. (27) May 13.
Economics of Medium-Sized Power Stations; a Study of Comparisons between Steam, Gas and Oil Engines.* A. J. J. Pfeiffer. (Abstract of paper read before the Inst. of Elec. Engrs.) (73) May 14.
The "Z" Electric Lamp Works, Southfields.* (26) May 14.
Divided-Fall Hydro-Electric Developments on the Hoosic River.* (27) May 20.
The Switchboard Equipment of the Indiana Steel Company at Gary, Ind.* E. A. Lof. (27) May 20.
System of the Philadelphia Electric Company.* (27) May 27.
Heavy Wire Terminations.* (27) May 27.
Output and Regulation in Long-Distance Lines. Percy H. Thomas. (42) June.
Calculation of the High-Tension Line. Percy H. Thomas. (42) June.
The Resistance and Reactance of Armored Cables. J. B. Whitehead. (42) June.
Reduction in Capacity of Polyphase Motors due to Unbalancing in Voltage.* S. B. Charters, Jr., and W. A. Hillebrand. (42) June.
Calculation of Iron Losses in Dynamo Electric Machinery.* I. E. Hanssen. (42) June.
Comparative Costs of 25-Cycle and 60-Cycle Alternators.* Carl J. Fechheimer (42) June.
The Heating of Induction Motors.* A. Miller Gray. (42) June.
Function of Fly-Wheels in Connection with Electrically Operated Rolling Mills.* H. C. Specht. (42) June.
Repulsion Motor with Variable-Speed Shunt Characteristics.* E. F. W. Alexanderson. (42) June.
The Measurement of the Insulation Resistance of a Live Three-Wire System.* Gisbert Kapp and J. Dennis Coales. (77) June.
Experimental Analysis of Armature Reaction.* Gisbert Kapp. (77) June.
The "G. B." System from a Tramway Manager's Point of View.* Stanley Clegg. (77) June.
Parallel Operation of Alternators.* E. Rosenberg. (77) June.
The Improvement of Power Factor in Alternating-Current Systems.* Miles Walker. (77) June.
The Electro-Hydraulic Pumping Plant at the Bristol Docks.* W. A. Clatworthy. (77) June.
The Arrangement of Underground Conduit for Electrical Distribution. Sydney W. Ashe. (27) June 3.
Differential Ballistic Methods of Measuring Hysteresis Losses.* R. Beattie and P. M. Elton. (73) Serial beginning June 4.

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Electrical—(Continued).

Note on Horizontal Receivers and Transmitters in Wireless Telegraphy.* H. M. Macdonald. (Abstract of paper read before the Royal Soc.) (73) June 4.
The Function of the Earth in Radiotelegraphy.* J. A. Fleming. (11) Serial beginning June 4.
The Melbourne City Electricity Undertaking.* J. H. Davies. (26) June 4.
Central-Station Practice and Results from Waterloo, Ia.* (27) June 10.
Power Plant of University of Washington, Seattle, Wash.* George H. Tinker. (27) June 10.
Researches in Radiotelegraphy.* J. A. Fleming. (Abstract of paper read before the Royal Inst.) (73) Serial beginning June 11; (11) Serial beginning June 11.
Electricity and the "Times." (73) June 11.
The Comparative Values of Various Coatings and Coverings for the Prevention of Soil and Electrolytic Corrosion of Iron Pipe. Robert B. Harper. (Abstract of paper read before the Ill. Gas Assoc.) (83) June 15.
High-Tension Transmission System at Gaucin-Seville, Spain.* (27) June 17.
Recent Improvements in Underground Conduit Construction.* S. B. Way. (27) June 17.
Cost of Underground Conduit Systems for Electrical Distribution.* Sydney W. Ashe. (27) June 17.
The New Telefunken System.* Count Arco. (Abstract of paper read before the Ver. Deutsch. Elek.) (73) June 18.
The Lepel Wireless Telegraph System.* J. Erskine-Murray. (73) June 18.
Electric Power at Messrs. Cassell & Co.'s Printing Works. (26) June 25.
Method of Testing Transformer Core Losses, giving Sine-Wave Results on Commercial Circuits.* L. W. Chubb. (42) July.
A Sketch of the Theory of the Adjustable-Speed, Single-Phase, Shunt Induction Motor.* F. Creedy. (42) July.
Surges on a Cable System with an Aluminum Surge Protector.* E. E. F. Creighton and S. D. Sprong. (42) July.
The Application of Storage Batteries to the Regulation of the Alternating-Current Load at the Plant of the Indiana Steel Company, Gary, Indiana.* J. Lester Woodbridge. (42) July.
Some Considerations in Designing Heavy Capacity Fuses.* Louis W. Downes.
The Modern Telephone Cable.* Frank B. Jewett. (42) July.
Even Harmonics in Alternating-Current Circuits.* John B. Taylor. (42) July.
Multispeed Induction Motors.* H. G. Reist and H. Maxwell. (42) July.
Electrical Measurements on Circuits Requiring Current and Potential Transformers.* L. T. Robinson. (42) July.
Automatic Motor Control.* H. E. White. (42) July.
The Testing of Transformer Steel. M. G. Lloyd and J. V. S. Fisher. (42) July.
The Convection of Heat from Small Copper Wires.* A. E. Kennelly, C. A. Wright and J. S. Van Bylevelt. (42) July.
The Current Locus of the Single-Phase Induction Motor.* A. S. Langsdorf. (42) July.
Corona Phenomena in Air and Oil and their Relation to Transformer Design.* W. S. Moody and G. Faccioli. (42) July.
Improvement of Power-Factor and Commutation Conditions in Single-Phase Series Motors.* A. S. McAllister. (3) July.
Electromotive Force Wave-Shape in Alternators.* Comfort A. Adams. (42) July.
Plant of the Greenville Light, Water & Ice Company, Greenville, Ky. James S. Knowlson. (27) July 1.
Conditions Encountered by Towers in High-Tension Transmission Lines. N. J. Neall. (27) July 1.
The Construction of High-Frequency Apparatus.* George T. Hanchett. (27) July 1.
Modern Cable Systems.* E. M. Hollingsworth. (Paper read before the Inc. Mun. Elec. Assoc.) (47) July 2.
The New Telefunken Apparatus.* (73) July 2.
The Public Supply of Electric Power. G. L. Addenbrooke. (29) July 9.
Cheap Units.* Alex. Sinclair. (Paper read before the Inc. Mun. Elec. Assoc.) (47) July 9.
Experimental Method for the Analysis of E. m. f. Waves.* P. G. Agnew. (27) July 15.
The Electric Power Supply of Chicago.* (73) Serial beginning July 16.
Antennæ. G. W. Pickard. (Abstract of paper read before the Wireless Inst.) (73) July 23.

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12 500-H. P. Turbines of the *North Dakota*.* (64) May 25.
Some Experiments with Lightened Beam Brackets.* R. Earle Anderson. (95) Serial beginning June.

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Marine—(Continued).

Mast and Derrick Mountings.* **(95)** Serial beginning June.
Transmission of Intelligence on Steam Vessels.* H. A. Hornor. **(3)** June.
The French Submarine Salvage Boat *Vulcain.** **(12)** June 11.
The Scantlings of Steel Vessels. **(11)** June 25; **(12)** June 25.
The Operation and Management of the Parsons Marine Steam Turbine as Practised on the U. S. S. *Chester.** A. F. H. Yates. (From *Journal,* Am. Soc. Naval Engrs.) **(12)** Serial beginning June 25.
The Shipbuilding and Engineering Company of Akers Mekaniske Verksted.* A. Gundersen. **(95)** July.
Superheated Steam in Marine Work.* F. J. Rowan. **(95)** Serial beginning July.
The Marine Steam Engine Indicator.* Charles S. Root. **(95)** Serial beginning July.
The Management and Equipment of an Experimental Tank. **(12)** Serial beginning July 16.
Reinforced Concrete Boats.* H. Prime Kieffer. **(95)** Aug.
A 6 000-Ton Floating Drydock.* William T. Donnelly. **(95)** Aug.
Some Model Experiments on Suction of Vessels.* D. W. Taylor. (Paper read before the Amer. Soc. Naval Archts. and Mar. Engrs.) **(95)** Aug.

Mechanical.

Some Modern Retort Coke Ovens, with Special Reference to the Practice of the Nova Scotia Steel and Coal Company, Limited, at Sydney Mines, N. S.* C. L. Cantley. **(5)** Vol. 22, Pt. 2.
Some Considerations in the Application of Low Pressure Steam Turbines to Power Generation.* J. R. Bibbins. **(5)** Vol. 22, Pt. 2.
The Central Power-Station of the De Beers Consolidated Mines, Ltd., Kimberley, South Africa.* Percy A. Robbins. **(56)** Vol. 39.
Typecasting and Composing Machinery.* L. A. Legros. **(75)** Dec., 1908.
The Development of the Downflow Boiler.* J. C. Parker. **(2)** Jan.
The Value of Gas Power.* Charles E. Lucke. **(6)** Apr.
Alloy Steels for Motor-Car Construction. John A. Mathews. **(3)** May.
Results with Vertical Retorts at Trieste.* E. Sospisio. (From *Il Gaz.*) **(66)** May 4.
Reversing Valve Gears in General Use.* Simpson Rice. **(64)** May 11.
Bolz's Arrangement of Vertical Retorts.* **(66)** May 11.
Calorific Value of California Fuel Oil. R. W. Fenn, M. Am. Soc. C. E. **(13)** May 13.
Power from the Sun's Heat.* **(13)** May 13.
Experiments in the Development of Power from the Sun's Heat.* H. E. Willsie. **(13)** May 13.
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Indicator Diagrams.* W. A. Tookey. (Paper read before the Assoc. of Engrs.-in-Charge.) **(47)** Serial beginning May 14.
Gyroscopic Balancing of Aeroplanes.* Lucien Fournier. **(19)** May 15.
Critical Economies in Portland Cement Production.* E. A Richmond. **(14)** May 15.
Calorimetry.* J. B. Klumpp. (Paper read before the New Eng. Assoc. of Gas Engrs.) **(24)** May 17; **(83)** May 15.
Mechanical Equipment of the Plaza Hotel.* Warren O. Rogers. **(64)** May 18.
Permanent Molds for Iron Castings.* Edgar A. Custer. **(20)** May 20.
The Computation of Engine Horse-Power Rating Formulæ.* W. Worby Beaumont. (Report to the Royal Automobile Club.) **(12)** May 21.
American Balanced Valves.* **(15)** May 21.
Aeroplane Problems. Herbert Chatley. **(19)** Serial beginning May 22.
Cast-Iron Molds and Their Effect upon Castings made in Them. Edgar A. Custer. (Abstract of paper read before the Amer. Foundrymen's Assoc.) **(13)** May 27.
Variation in Angular Velocity of Fly-Wheels.* Alex. R. Horne, A. M. I. Mech. E. **(11)** May 28.
The Purchase of Fuel on a British Thermal Unit Basis.* Lawrence P. Crecelius. **(42)** June.
Smoke Prevention or Smoke Consumption.* Harold V. Coes. **(9)** Serial beginning June.
A Suggestion for Overloaded Boiler Plants.* Carl S. Dow. **(9)** June.
Mill B of the Pacific Portland Cement Company, Con., near Suisun, Cal.* **(67)** June.
Boiler Corrosion as an Electrochemical Action.* C. F. Burgess. **(4)** June.
The Suction Gas Producer.* T. W. Burt. **(10)** June.
Clutches.* Thomas R. Shaw, A. M. I. M. E. **(47)** Serial beginning June 4.
Easement Curves.* Robert H. Smith. **(12)** June 4.
The Thermal and Combustion Efficiency of a Petrol-Motor.* **(11)** June 4.
The Davidson "Gyropter" Flying Machine.* **(19)** June 5.

*Illustrated.

Mechanical—(Continued).

Design of Condenser and Cooling Tower Plants. M. R. Bump. (Abstract of paper read before the Nat. Elec. Light Assoc.) (14) June 5.

The Comparative Practical Efficiency of Various Types of Gas Lamp. W. C. Ware. (Paper read before the New England Assoc. of Gas Engrs.) (24) June 7; (83) July 1.

A Comparison of the Enriching Values of Benzole and Gas Oil.* L. J. Willien. (Paper read before the New England Assoc. of Gas Engrs.) (24) June 7; (83) July 1.

The Relative Advantages of Reciprocating Engines and Turbines for Traction Generating Stations.* G. Rizzo. (Abstract of a Report to the Soc. Gen. des Ch. de F. economiques.) (73) June 11.

300-Ton Testing Machine at Birmingham University.* (22) June 11.

Vacuum Ash Conveyor at Armour Glue Works.* George B. Hess. (64) June 15.

Schoen Steel Wheels.* (20) June 17.

Pistons and Piston Valves.* (12) Serial beginning June 18.

The Mechanics of Ilgner-Operated Winding-Engines.* R. H. Collingham, A. M. Inst. C. E. (11) June 18.

Low-Pressure Steam Turbines.* C. H. Smoot. (Paper read before the National Elec. Light Assoc.) (64) June 22.

Sioux Falls Hydroelectric Development.* Simpson Rice. (64) June 22.

Gas Heating Research. (Report of a Committee to the Inst. of Gas Engrs.) (66) June 22.

Carbonization in Chamber Settings.* R. Lessing. (Paper read before the Inst. of Gas Engrs.) (66) June 22.

Carbonizing.* J. Ferguson Bell, M. Inst. C. E. (Paper read before the Inst. of Gas Engrs.) (66) June 22.

Some Advantages and Disadvantages of a Hot-Coke Conveyor.* Robert Watson. (Paper read before the Inst. of Gas Engrs.) (66) June 22.

Illuminating Efficiencies of Carbon Monoxide and Hydrogen used in Conjunction with Incandescent Mantles.* Arthur Forshaw. (Paper read before the Inst. of Gas Engrs.) (66) June 22.

A Study in Working Costs (Gas). Herbert Lees. (Paper read before the Inst. of Gas Engrs.) (66) June 22.

The Harmet Process of Ingot Compression.* J. F. Springer. (20) June 24.

Tests of a Double-Zone Bituminous Gas Producer.* John B. Klumpp, Assoc. Am. Inst. E. E.; Carlton A. Graves, Assoc. Am. Inst. E. E., and Irving E. Moultrop, M. Am. Soc. M. E. (Abstract from report to the National Elec. Light Assoc.) (13) July 1; (14) June 26; (20) June 17.

The Absorption Refrigerating Machine.* W. E. Crane. (64) June 29.

Gasoline Electric Automobiles.* Justus B. Entz. (3) July.

The Advantages and Disadvantages of Motor Omnibuses. Results of Working. Mauclère. (Report to the Inter. Tramways and Light Rys. Union.) (88) July.

The Advantages and Disadvantages of Motor Omnibuses. Results of Working. K. Otto. (Report to the Inter. Tramways and Light Rys. Union.) (88) July.

Salt Glazing. A. F. Greaves-Walker. (76) July.

The Dirigible Baloon with Gyroscope Control. Russell Thayer. (3) July.

Recent Developments in Large Gas-Engine Design.* Percy R. Allen. (10) Serial beginning July.

Aerial Tramway for Coal.* Robert M. Magraw. (45) July.

The Diagonal Patch (Boilers).* R. E. M'Namara. (94) July.

Proposed I. M. B. M. A. Standard Punches, Dies and Couplings.* (94) July.

Power Requirements for Rolling High-Carbon Steel of Small Section.* Brent Wiley. (42) July.

Electric-Driven Rolling Mills. E. Friedlander. (42) July.

Freight-Handling Equipment; an Analysis of Modern Practice. Werner Boecklin. (9) July.

Recent Development in Conveying Machinery for Coal and Ashes.* Wilbur G. Hudson. (9) July.

Koppers Ammonia Sulphate Plant.* (83) July 1.

The Koppers By-Product Coke Oven.* (57) July 2.

The Relation of the Character of Coals to the Prevention of Smoke. D. T. Randall. (Paper read before the Inter. Assoc. for the Prevention of Smoke.) (96) July 2; (62) July 12; (14) July 3.

The First Installation of Dessau Vertical Retorts in England.* (66) July 6.

A System of High and Low Pressure Hydraulic Accumulators.* R. F. Williams. (15) July 9.

The Vertical Retort Installation under the Dessau System in Course of Construction for the Providence Gas Co., Providence, R. I.* Frederick J. Mayer. (83) July 15.

The Measurement of Compressed Air.* J. Orr. (Paper read before the Transvaal Inst. of Mech. Engrs.) (47) Serial beginning July 16.

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Solution of Flight Problems by Graphical Methods.* M. F. Logan. (12) July 16
The Pyrometry of the Beehive Coke Oven.* J. R. Campbell. (Paper read before the Coal Min. Inst. of Amer.) (16) July 17.
Heat Transfer (Boilers).* (Extracts of paper read before the Amer. Ry. Master Mech. Assoc.) (18) July 17.
The Development of the Ammonia Industry in Gas Works during the Last Fifty Years.* R. W. Hilgenstock. (24) July 19.
The Wetterhorn Cableway Incline.* (13) July 22.
The Virginian Railway's Coal Handling Plant, Sewell's Point, Va.* (20) July 22.
Sellers 42-In. Car Wheel Lathe.* (15) July 23.
A Fatigue Testing Machine.* J. H. Smith. (11) July 23.
Experience with Suction Gas Producers. (12) July 23.
New Stone-Crushing Plant of the Dolese & Shepard Company.* (14) Serial beginning July 24.
The Selection of an Oil for Lubrication.* Henry M. Wells and Wm. Scott-Taggart. (64) July 27.
Plant of the Commonwealth Power Company.* Osborn Monnett. (64) July 27.
The Mechanical Plant of the Harvard Medical School.* Howard S. Knowlton. (14) Serial beginning July 31.
Wetterhorn Aerial Cableway.* Wm. F. Johnston. (14) July 31.
Belt Conveyors for Handling Materials.* George Frederick Zimmer, A. M. Inst. C. E. (10) Aug.
The Gasification of Peat with Recovery of Chemical By-Products. Charles A. Davis. (10) Aug.
Recent Developments in Conveying Machinery for Coal and Ashes.* Wilbur G. Hudson. (9) Aug.
Etat Actuel de l'Aviation.* Taris. (43) Mar.
De la Compression de la Vapeur Appliquée à l'Evaporation.* Jean Rey. (32) Apr.
Application de la Compression de la Vapeur à l'Evaporation des Liquides Industriels.* Ch. Prache et Ch. Bouillon. (32) Apr.
Sur la Cémentation du Fer et de Ses Alliages par l'Oxyde de Carbone. G. Charpy. (93) May.
Essais Exécutés sur une Nouvelle Mi-Fixe Compound-Tandem à Surchauffe, la Maison R. Wolf.* Gutermuth. (37) May.
Le Transbordeur Funiculaire à Voyageurs du Mont Ulia, près de Saint-Sébastien (Espagne).* G. Espitalier. (33) June 5.
Les Moteurs à Explosion Légers pour Dirigeables et Aéroplanes.* Ch. Dantin. (33) Serial beginning June 5.
Détermination du Diamètre et du Poids des Roues des Automobiles.* E. Girardault. (33) July 10.
La Condensation par Mélange système Westinghouse-Leblanc, à la Fosse Lambrecht, des Mines d'Anzin.* Ch. Dantin. (33) July 17.
Neue Methoden zur Berechnung von Kalibrierungen.* W. Tafel. (50) May 5.
Die Blechwalzwerksanlage der Mossend Steel Works in Mossend bei Glasgow.* (50) May 12.
Teer- und Ammoniakbehälter des Gaswerkes der Stadt Weimar.* K. Schaaf. (78) May 15.
Untersuchungen über den Arbeitsvorgang im Injektor.* G. Schrauff. (48) Serial beginning May 15.
Der Kraftbedarf von Walzwerken.* H. Groeck. (48) May 22.
Die Neuen Walzwerksanlagen der Westfälischen Stahlwerke in Bochum.* (50) May 26.
Ueber den Schnittwiderstand bei der Bearbeitung der Metalle durch Abheben von Spänen. H. Friedrich. (48) May 29.
Die van der Waalssche Theorie der Gase und Flüssigkeiten und Ihre Beziehungen zu den Ingenieurwissenschaften. J. B. Goebel. (48) May 29.
Eine Neue Sandaufbereitung.* C. Henning. (Paper read before the Deutsche Giessereifachleute.) (50) June 2.
Die Elektrisch Betriebene Umkehrblockstrasse der Rheinischen Stahlwerke.* Georg Meyer. (50) June 9.
Geradbahn- und Kreisbahn-Beizmaschinen.* U. Lohse. (50) Serial beginning June 17.
Bremsresultate einer Kesselturbine von 300 PS.* Karl Gottwein. (53) June 18.
Die Anlagen der Oberschlesischen Eisenbahn-Bedarfs-A.-G. zu Friedenshütte.* (50) June 23.
Turbodynamos und Turboelektromotoren.* F. Niethammer. (48) (Serial beginning June 26.
Die Regelung von Dampfturbinen und ihr Einfluss auf die Leistungsentwicklung in den einzelnen Druckstufen. Herbert Baer. (48) Serial beginning July 3.
Die Leistungen von Kreiseln und Schöpfrädern. H. Hagen. (48) Serial beginning July 3.
Kammlagerkonstruktionen sowie deren Berechnung bei Turbomaschinen mit horizontaler und vertikaler Welle.* Emil Gutmann. (97) July 5.

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Mechanical—(Continued).

Kritik der Drachenflieger.* Ansbert Vorreiter. (48) Serial beginning July 10.
Massenbeförderung von Kohle auf den Gruben der Anhaltischen Kohlenwerke.* E. Ullmann. (48) July 17.

Metallurgical.

The Air Furnace Process of Preparing White Cast-Iron for the Malleablizing Process.* Henry M. Howe. (56) Vol. 39.
The Wilfley Table, II. Robert H. Richards. (56) Vol. 39.
Charcoal and Coke as Blast-Furnace Fuels. R. H. Sweetster. (56) Vol. 39.
The Carbon-Iron Diagram.* Henry M. Howe. (56) Vol. 39.
The Mechanical Preparation of Ores in Sardinia.* Erminio Ferrario. (56) Vol. 39.
The Hardinge Conical Pebble-Mill.* H. W. Hardinge. (56) Vol. 39.
Investigations on Jigging.* Royal Preston Jarvis. (56) Vol. 39.
The Relation of Slow Driving to Fuel-Economy in Iron Blast-Furnace Practice. John B. Miles. (56) Vol. 39.
The Treatment of the Gold-Ores of Hog Mountain, Alabama. T. H. Aldrich, Jr. (56) Vol. 39.
The Constitution of Copper-Iron and Copper-Lead-Iron Mattes.* Charles H. Fulton and Ivan E. Goodner. (56) Vol. 39.
The Behavior of Calcium Sulphate at Elevated Temperatures with Some Fluxes.* H. O. Hofman and W. Mostowitsch. (56) Vol. 39.
Gayley's Invention of the Dry Blast. R. W. Raymond. (56) Vol. 39.
Experience with the Gayley Dry Blast at the Warwick Furnaces, Pottstown, Pa.* Edward B. Cook. (56) Vol. 39.
The Pearce Gold-Separation Process. Harold V. Pearce. (56) Vol. 39.
The Magnetic Separation of Ores and Its Application to Spathic Iron (Siderite) in Siegerland.* W. J. Bartsch. (Abstract of paper read before the Assoc. Berggeist.) (22) May 7.
Stock Distribution for Blast Furnaces.* J. E. Johnson, Jr. (20) May 13.
The Heroult Electric Steel Furnace.* Robert Turnbull. (Paper read before the Amer. Electrochemical Soc.) (20) May 13.
On Charcoal Blast Furnace Practice in the Ural.* M. A. Pavloff. (12) May 14.
Improvements in the Treatment of Slime by the Vacuum Filter Process.* Arthur W. Allen. (16) May 15.
The Electric Furnace Reduction of Iron Ore. Joseph W. Richards. (Paper read before the Amer. Electrochemical Soc.) (13) May 20.
The Pierce Amalgamator.* Justin H. Haynes. (45) June.
The Ohio Concentrator; a Description of the Mill at Bingham, Utah.* Leroy A. Palmer. (45) June.
Researches on the Magnetic Properties of a Series of Carbon Tungsten Steels.* Thomas Swinden. (77) June.
A Contribution to the Study of Electric Furnaces as Applied to the Manufacture of Iron and Steel.* Ch. A. Keller. (Abstract of papers read before the Faraday Soc. and the Amer. Electrochemical Soc.) (22) June 18; (73) July 2; (20) June 3.
Automatically Circulating Furnaces of the Gin Type for the Electrical Production of Steel.* Gustave Gin. (Abstract of paper read before the Faraday Soc.) (22) June 18.
The Kilker Matte Tapping Car.* F. T. Havard. (16) June 26.
The Tintic Smelter.* Leroy A. Palmer. (45) July.
Pittsburg Silver Peak Mill.* Henry Hanson. (45) July.
The Gayley Dry Air Blast. (3) July.
Methods of Refining Steel in the Electric Furnace.* J. B. C. Kershaw. (10) July.
Designing Blast Furnaces.* M. Pavloff. (22) July 9.
An All Slime Cyanide Plant at Guanajuato, Mexico. Edwin Shapley. (16) July 10.
Tailings Disposal Plant at the Wolverine Mill.* C. Kemble Baldwin. (16) July 10.
Shortening the Roasting Period for Mercury Ores.* William B. Dennis. (16) July 17.
Determining Carbon in Steel.* Warren I. Keeler. (20) July 22.
Recherches Expérimentales sur la Nature des Mattes de Cuivre.* A. Baykoff et N. Troutneff. (93) May.
Quelques Expériences sur la Trempe du Plomb et les Changements Corrélatifs de sa Duretée.* Loutchinnsky. (93) May.
Les Alliages de Cuivre Spéciaux.* Jean Escard. (33) Serial beginning May 22.

Military.

Field Guns for Destroying Dirigibles.* (19) May 29.
The Engineering of Ordnance.* A. Trevor Dawson. (Paper read before the Jun. Inst. of Engrs.) (11) Serial beginning July 2.

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Mining.

The Mining and Milling of Silver-, Lead- and Zinc-Ores at Pierrefitte Mines, France. William Waters Van Ness. (56) Vol. 39.
Sulphur Dioxide as an Agent in Fighting Mine-Fires. Walter O. Snelling. (56) Vol. 39.
Calculation of Mine Values. R. B. Brinsmade. (56) Vol. 39.
Effect of Humidity on Mine Explosions. Carl Scholz. (56) Vol. 39.
Requirements of a Breathing-Apparatus for Use in Mines.* Walter E. Mingramm. (56) Vol. 39.
The Ilsede Hütte Iron-Mines at Peine, Germany.* Lucius W. Mayer. (56) Vol. 39.
Machine Mining under Difficulties.* John Gibson. (59) Vol. 31, Pt. 5.
Dredging at Butte.* A. Floyd Bushnell. (16) May 15.
New Method of Extracting Oil from Boreholes.* Frederick A. Talbott. (16) May 15.
A Comparison of Coal-Cutting Machines. Sydney F. Walker. (16) May 22.
Electricity in Coal Mines. Robert Nelson. (Paper read before the Inst. of Min. Engrs.) (22) May 28; (57) June 4.
The Use of Concrete for Mine Support. W. R. Crane. (Paper read before the Inst. of Min. Engrs.) (22) May 28.
The Coal Dust Question in Great Britain.* Henry Hall. (16) May 29.
Suction Gas and Its Application to Mining.* George D. Stephen. (16) May 29.
New River Coal Field, W. Va.* H. H. Stoek. (45) June.
The Steel Oil Derrick.* R. B. Woodworth. (58) June.
A Model Coal Mine in Westphalia.* William S. Hall. (16) June 5.
The Panel System as Applied to Metal Mining.* H. E. West. (16) June 12.
A Novel Coal and Stone-Cutting Process.* Alfred Gradenwitz. (16) June 19.
Automatic Dumping Skip for Vertical Shafts.* Geo. C. McFarlane. (16) June 26.
Ray Consolidated Mines.* R. L. Herrick. (45) July.
Determination and Regulation of the Percentage of Gas in Mine Air-Currents.* John Gordon Smyth. (Paper read before the West Va. Coal Min. Inst.) (45) July; (16) July 3.
Some Simple Causes of Mine Explosions.* L. D. Tracy, Assoc. M. Am. Soc. C. E. (13) July 1.
Survey to Fix Position of the Water Haigh Pits. G. R. Thompson. (Paper read before the National Assoc. of Colliery Mgrs.) (22) July 2.
The Experimenting Station at Lievin, France.* Tony Callot. (16) July 3.
An Electrically Operated Hoisting Plant.* A. Gradenwitz. (16) July 10.
Electric Hoisting Equipment at Winona, Mich.* R. R. Seeber. (16) July 17.
Pumping Plant at the Tombstone Consolidated.* Elton W. Walker. (16) July 24.
Modern Gold Placer Dredging.* George B. Massey, 2d. (10) Aug.
La Lutte contre les Poussières de Houille dans les Mines.* J. Daniel. (Abstract from *Annales des Mines.*) (33) July 17.

Miscellaneous.

Notes on Canadian Forestry. Stanislas Gagné. (5) Vol. 22, Pt. 2.
Chemistry and the Conservation of our Forests and Minerals. Marston Taylor Bogert. (6) Apr.
The Salette Landslide of 1908 and some Earlier Quebec Landslides.* (13) May 27.
Temperature Determination and Control for High-Speed Tool Treatment (Pyrometers).* O. M. Becker. (9) June.
Going Value and Methods for its Computation.* John W. Alvord. (Abstract of paper read before the Amer. W. W. Assoc.) (14) June 19.
Difficult Problems which Public Utility Commissions are Endeavoring to Solve. H. C. Abell. (From *The Armour Engineer.*) (24) Serial beginning June 21.
Cost Estimating; a Discussion of Principles with Actual Estimates for Contract Work. Jean Bart Balcomb. (86) July 14.
La Radioactivité en Géologie et dans l'Atmosphère. P. Besson. (32) Mar.
Etude des Vibrations du Sol lors des Tremblements de Terre Conséquences au Point de Vue Théorique.* J. Bergeron. (32) Mar.
L'Or à Madagascar. Ch. Marquet. (32) Apr.
Les Anciennes Charrues de France.* Henri Chevalier. (32) May.

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The Rotherhithe Tunnel (Street traffic).* Edward Henry Tabor. (63) Vol. 175.
The First International Road Congress at Paris in October, 1908; what Led up to it; its Deliberations; and its Results. Austin B. Fletcher. (Paper read before the Bost. Soc. Civ. Engrs.) (1) Apr.
Road Machinery.* A. Rankin. (Paper read before the Ont. Good Roads Assoc.) (96) May 21.
Concrete Roadways. Ernest McCullough. (96) May 28.
Concrete Roadways.* Charles W. Ross. (96) June 4.
Vitrified Brick Roadways.* Will P. Blair. (From *Good Roads Magazine.*) (76) July.

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An "Accelerated Test" of Road Wear by Automobile Traffic.* (13) July 8.
The Public Lighting of the City of London; Report of the Deputation to Continental Cities. (66) July 20.
Road Materials and Some Simple Rules for Testing Them. Austin B. Fletcher, M. Am. Soc. C. E. (Paper read before the 1st Cong. of Road Builders.) (14) July 31.
Les Plantations Routières. Edmond Hermans. (30) June.
Die Bisherigen Ergebnisse der Teerungen auf den Rheinischen Provinzialstrassen bis zum Frühjahr, 1908. Görz. (Paper read before the Architekten- und Ingenieur-Verein zu Cassel.) (51) May 8.

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On the Serviceable Life and Average Annual Cost of Locomotives in Great Britain. Richard Price-Williams, M. Inst. C. E. (63) Vol. 175.
Glasgow Central Station Extension.* Donald Alexander Matheson, M. Inst. C. E. (63) Vol. 175.
The Rotherhithe Tunnel (Street traffic).* Edward Henry Tabor. (63) Vol. 175.
Straight Air Brake Equipment.* J. B. Parham. (5) Vol. 22, Pt. 2.
Improvements at the Works of the Canadian Locomotive Company, Kingston, Ont. Henry Goldmark. (5) Vol. 22, Pt. 2.
A Calibrated Speed Recorder (Train movement).* A. B. Stitzer. (2) Jan.
Influence of Ash on Value of Coal in Locomotive Service.* (61) Apr.
The Resistance of Railway Trains. Albert Frank. (88) May.
Uniformity in the Symbols used to Represent Different Types of Locomotives. P. Labryn. (From *De Ingenieur.*) (88) May.
Railroad Terminal Improvements at Providence, R. I.* George B. Francis. (Paper read before the Boston Soc. of Civ. Engrs.) (1) May.
The East Side Tunnel and its Approaches, Providence, R. I.* Edwin P. Dawley. (Paper read before the Boston Soc. of Civ. Engrs.) (1) May.
Development in Air Brakes for Railroads with a Brief Review of Past and Present Operating Conditions. W. V. Turner and S. W. Dudley. (65) May.
New 4-4-0 Engines, North-Eastern Railway.* (15) May 14.
Shop Notes from the C., St. P., M. & O. (15) May 14.
Sudbury-Kleinburg Branch of the Canadian Pacific.* (15) May 14.
Reinforced Concrete Roundhouse, A., T. & S. F. Ry.* (18) May 15.
Conversion of New Canaan Branch from 500-Volt D. C. to 11 000-Volt A. C. Operation.* (17) May 15.
Electric Train Staff on the Southern Pacific.* (15) May 21.
A Suggestion for Tie Rods.* H. Herden. (15) May 21.
Railway Development in China. Arthur John Barry. (29) May 21.
The Protection of Grade Crossings.* (96) May 21.
Comparative Rail Specifications. (15) May 21.
New York Passenger Terminal, Central R. R. of New Jersey.* (14) May 22.
Steel Catenary Bridges for Lines in Central New York.* (17) May 22.
The Sigafoos Tunnel Machine.* Rowland Ashford Phillip. (46) May 22.
Methods of Snow-Fighting on Railroads. A. W. Wheatley. (Paper read before the Canadian Ry. Club.) (86) May 26.
Hydraulic Construction of Large Embankments on the Chicago, Milwaukee & Puget Sound Ry.* (13) May 27.
The Jacobs-Shupert Locomotive Firebox.* (15) May 28.
The Passenger Rate of the American Railway. William S. Bronson. (15) Serial beginning May 22.
Mikado Type Locomotives Built by C., M. & St. P. Ry.* (18) May 29.
The Waterbury Improvements of the New Haven Railroad.* (14) May 29.
Chicago Track Elevation.* M. K. Trumbull. (15) June 4; (14) May 29; (13) June 3.
Mallet Articulated Compound Locomotives 2-6-6-0 Type, Virginian Railway.* (25) June; (15) June 18; (18) June 12.
Baker-Pilliod Valve Gear.* (21) June.
The Manufacture of Points and Crossings.* (21) Serial beginning June.
Zara's Bogie; Italian State Railways.* (21) June.
The Question of the Proposed Trials of Goods-Train Brakes. A. Huberti and J. Doyen. (88) June.
The Resistances and the Application of Double-Wire Transmissions for Operating Switches and Signals at a Distance.* L. H. N. Dufour. (88) June.
Low, Moderately and Highly Superheated Steam (Locomotives).* F. J. Cole. (25) June.
Mikado Type Freight Locomotive, Virginian Railway.* (25) June.
Location of Side Bearings on Tender Trucks.* Theodore H. Curtis. (25) June.
Self Clearing Ash Pans (Locomotives).* (25) June.
Flange Wear of Driving Tires (Locomotives).* R. H. Rogers. (25) June.

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Comparative Tests of Water Tube and Standard Fireboxes.* S. S. Riegel. (25) June.
Cylinders for 4-Coupled Compound 2-6-4 Engines; Austrian State Railways.* (21) June.
Mechanical Appliances for Track-Laying.* J. F. Springer. (10) June.
Some Remarkable Locomotives of 1908* J. F. Gairns. (10) June.
Details of Articulated Mallet Compound Locomotive for the Southern Pacific.* (15) June 4; (94) July.
Dimensions of New Locomotives. (96) June 4.
The Construction of the Transandine Railway.* (14) June 5.
Ten-Wheel Tank Locomotive (3-Ft. 6-In. Gauge); Dutch Indian Railway.* (11) June 11.
The Electrification of the L. B. & S. C. Railway's South London Line.* (73) June 11.
The Semaphore; Upper Left vs. Upper Right.* C. H. Morrison. (Abstract of paper read before the Ry. Signal Assoc.) (15) June 11; (18) June 26.
Simmen Automatic Railway Signal Device.* (96) June 11.
The Campbell-Olden Steel Side-Dump Car.* (18) June 19.
Locomotive Road Tests of Briquets. C. T. Malcolmson. (18) Serial beginning June 12.
Overhead Construction for the Electrification of Main Trunk Lines.* Francis S. Denneen. (Extracts from *Purdue Eng. Review.*) (8) June 12.
The Signal and Dispatching System on the Mimico Division of the Toronto & York Radial Railway.* (17) June 12.
A Locomotive Firebox without Staybolts in the Side Sheets and Crown Sheet (Atchison, Topeka & Santa Fe Ry.).* (13) June 17.
All-Electric Interlocking at Alientown.* (15) June 18.
Mechanical Stokers. (Report to the Amer. Ry. Master Mechanics Assoc.) (18) June 19.
New Locomotives for the St. Louis Southwestern Ry.* (18) June 19.
Detroit River Tunnel Locomotive.* (17) June 19; (13) June 24; (18) June 19.
New York Central Electric Locomotive.* (15) June 25.
Automatic Junction Signals at Calvin, Oklahoma.* (15) June 25.
Remarkable Plant of the St. Clair Tunnel.* Osborn Monnett. (64) June 29.
Pacific and Consolidation Type Locomotives, Chicago and Alton Railway.* (25) July; (18) June 5.
A Tabular Comparison of Notable Examples of Recent Locomotives, Arranged in Order of Total Weights. (25) July.
Locomotive Performance under Saturated and Superheated Steam.* W. F. M. Goss. (Abstract of paper read before the Master Mechanics' Assoc.) (25) July; (18) June 26.
On the Question of Improvements in Locomotive Boilers. Nadal. (88) July.
On the Question of Strengthening the Track and the Bridges with a View to Increasing the Speed of Trains. J. W. Jacomb-Hood. (88) July.
The "T. A. C." System of Electric Train Lighting.* (21) July.
Zara's Compensating Arrangement for Adjoining Axles.* (21) July.
On the Working of Italian Locomotives in Actual Service.* (21) July.
Some Comparisons of Wood and Steel Ties. J. M. Meade. (Paper read before the Roadmasters' Convention.) (87) July.
The Widening of Wemyss Bay Railway.* (11) Serial beginning July 2.
Consolidation Locomotives for the Pennsylvania.* (15) July 2.
The Loetschberg Tunnel.* H. Prime Kieffer. (15) July 2.
Grade Separation. F. L. Somerville. (96) Serial beginning July 2.
The Laying out of the Track in the Muskoka Divisional Yard.* C. D. Norton. (96) July 2.
Latest Developments in Electric Railway Apparatus. C. D. Eveleth. (Abstract of paper read before the St. Ry. Assoc. of N. Y.) (17) July 3.
Latest Improvements in Electric Railway Apparatus. J. L. Davis. (Abstract of paper read before the St. Ry. Assoc. of N. Y.) (17) July 3.
Construction Work on the C., M. & St. P. Coast Extension.* (18) Serial beginning July 3.
Splicing Sills.* (Abstract of report to Master Car Builders' Assoc.) (18) July 3.
Electrical Features of the New Northwestern Railroad Terminal in Chicago.* (27) July 8.
Complete Solution of Transmission Line with Distributed Capacity and Leakage.* Harold Pender. (27) July 8.
Special Motor-Controlling Devices in the Rolling Mill of the Indiana Steel Company, Gary, Ind.* (27) July 8.
Automatic Signal Records on the Baltimore & Ohio. (15) July 9.
Axle-Driven Head-End Electric Train Lighting System.* (15) July 9.
Compressed Gas and Electric Car Lighting.* Geo. L. Fowler. (15) July 9.
The Pressure on Locomotive Brake Blocks.* (12) July 9.

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Car Wheels.* (Report of a Comm. to the Master Car Builders Assoc.) **(18)** July 10; **(13)** July 29; **(25)** Aug.
Reinforced Concrete Culverts of the Schenectady Railway.* **(17)** July 10.
A Railway Transfer Table without a Pit.* Henry V. Miller. **(13)** July 15.
Wheel Foundry and Foundry Methods; Norfolk & Western.* George L. Fowler. **(15)** Serial beginning July 16.
The Effect of the Physical Characteristics of a Railway upon the Operation of Trains.* **(15)** Serial beginning July 16.
Opening of the Terminal Station of the Hudson & Manhattan Railroad.* **(17)** July 17.
The Earliest Balanced Locomotives.* Herbert T. Walker. **(15)** Serial beginning July 23.
Newburgh and North Fife Railway.* **(11)** July 23.
Equated Train Tonnage. J. M. Daly. (Abstract from *Ill. Central Employees Mag.*) **(18)** July 24.
Mallet Compound Locomotive for the Little River R. R.* **(18)** July 24.
Use of Alternating Currents in Railway Signaling. W. K. Howe. (Paper read before the Ry. Signal Assoc.) **(15)** Serial beginning July 9; **(18)** July 24.
The Operation of the Simplon Tunnel Railway.* **(13)** July 29.
Interlocking and Signalling.* E. W. Oliver. **(96)** July 30.
The Christiania-Bergen Railway.* **(15)** July 30.
Specifications for 90-Lb. Bessemer and Open Hearth Steel Rails for the Harriman Lines. **(15)** July 30.
Waterbury Extensions of the Connecticut Company.* **(17)** July 31.
Bank Versus Level Firing (Locomotives).* E. D. Nelson. (Paper read before the Am. Ry. Master Mechanics Assoc.) **(25)** Aug; **(18)** July 3.
Note sur un Procédé d'Etude des Formules d'Exploitation pour les Chemins de Fer d'Intérêt Local.* Tourtay. **(43)** May.
Système d'Essieux Coudés à Flasques Evidées (Locomotives).* Ch. Frémont. **(92)** May.
Note sur le Raccordement des Courbes et des Alignements dans les Voies de Chemins de Fer.* Collot. **(38)** May.
Chasse-Neige Rotatif à Vapeur de la Compagnie d'Orléans.* Conte. **(38)** May.
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MINUTES OF MEETINGS

OF THE SOCIETY

September 1st, 1909.—The meeting was called to order at 8.30 P. M.; Gardner S. Williams, Director, Am. Soc. C. E., in the chair; Chas. Warren Hunt, Secretary, and present, also, 113 members and 34 guests.

The minutes of the meetings of May 19th and June 2d, 1909, and of the Annual Convention, were approved as printed in *Proceedings* for August, 1909.

A paper by Howard J. Cole, M. Am. Soc. C. E., entitled "Concrete Piles," was presented by the author and illustrated with lantern slides. The paper was discussed orally by Messrs. M. M. Upson and Homer A. Reid, and the Secretary presented written communications on the subject from Messrs. Morton L. Tower, L. J. Mensch, Charles H. Higgins, and Thomas C. Desmond.

A paper by Cecil B. Smith, M. Am. Soc. C. E., entitled "Hydro-Electric Power in Canada," was presented by title, and the Secretary read a communication on the subject from Henry Holgate, M. Am. Soc. C. E.

The Secretary announced the election of the following candidates by the Board of Direction on August 31st, 1909:

As Members.

Manuel Alberto Coroalles, Havana, Cuba.
Bertram Dodd Dean, Seattle, Wash.
Joseph Oscar Eckersley, New York City.
Carl Leonard de Muralt, New York City.
Adelbert Franklin Parker, Ogden, Utah.
Franklin Calhoun Pillsbury, Boston, Mass.
John Richard Rablin, Boston, Mass.
Will Joseph Sando, Milwaukee, Wis.
Frank Scotten, Great Falls, Mont.
Robert Charles Strachan, New York City.
George Henry Tinker, Cleveland, Ohio.
Phillip Riley Van Frank, Jr., Little Rock, Ark.
Edward Franklin Vincent, Denver, Colo.
Frank Hiram Walker, Franklin, Pa.
Albert William Zesiger, Cleveland, Ohio.

As Associate Members.

Julius Andrew Arce, Arequipa, Peru.
Harry Edmund Bilger, Springfield, Ill.
Clifford Pinkney Bowie, San Francisco, Cal.
Glenn Smith Burrell, Honolulu, Hawaii.
Clyde Greyson Conley, Mt. Vernon, Ohio.
Arthur Garfield Crysler, Syracuse, N. Y.
Tranquilino Frasquieri y Regueifero, Havana, Cuba.
Ulysses S. Grant, 3d, Brookline, Mass.
William Bacon Gray, Stone Ridge, N. Y.
Russell Soule Greenman, Albany, N. Y.
William Henry Insley, Indianapolis, Ind.
Ralph De Witt Havens, Pawtucket, R. I.
Silas Bent Phillips, Seattle, Wash.
William Murray Pullar, Essendon, Victoria, Australia.
Rex Densmore Richardson, Scranton, Pa.
Estel Dean Rightmire, Atlantic City, N. J.
Henry Preston Rust, New York City.
Charles Wesley Schubert, Bellevue, Pa.
Ralph Carroll Soper, Powell, Wyo.
Myron Ellis Witham, Denver, Colo.

As Associates.

Albert John County, Philadelphia, Pa.
Harry Edward Jordan, Indianapolis, Ind.
Laurence Patrick Ryan, Chicago, Ill.

As Juniors.

Louis William Abrons, Manila, P. I.
George Crowell Andrews, Fulton, N. Y.
Carl August Bock, La Paz, Bolivia.
John Worde Caldwell, Honolulu, Hawaii.
John Wilbur Cunningham, Madison, Wis.
Lewis Repp Ferguson, Philadelphia, Pa.
Wilhelm Eduard Germer, Los Angeles, Cal.
Edgar Thomas Hayman, Annapolis, Md.
George Henry Knutson, Ann Arbor, Mich.
William Otto Lichtner, Newton Highlands, Mass.
John Nathaniel Mackall, Baltimore, Md.
Walden LeRoy Malony, Spokane, Wash.
James Gates Moore, Key West, Fla.
Leon Morley Pill, Jerome, Idaho.
James Gordon Steese, Culebra, Canal Zone, Panama.
Thomas King Sweesy, Berkeley, Cal.
Oliver Julian Todd, San Francisco, Cal.
Rene Barber Wright, Chicago, Ill.

The Secretary announced the transfer of the following candidates by the Board of Direction on August 31st, 1909:

From Associate Member to Member.

Edward Gatling Bradbury, Columbus, Ohio.
Charles Benedict Grady, West Orange, N. J.
Charles Davis Vail, Denver, Colo.
Joseph Wright, Glendive, Mont.

From Associate to Member.

Robert Fletcher, Hanover, N. H.

From Junior to Member.

Augustus Sayre Kibbe, Philadelphia, Pa.

From Junior to Associate Member.

Ralph Melvin Conner, Babb, Mont.
George Snyder Gildersleeve, Detroit, Mich.
Ralph Daniel Hayes, Stillwater, N. Y.
Linn Murdoch Huntington, Guantanamo, Cuba.

The Secretary announced the following deaths:

GEORGE H. BISHOP, elected Member August 7th, 1872; died August 20th, 1909.

LEFFERT LEFFERTS BUCK, elected Member February 3d, 1875; died July 17th, 1909.

GEORGE HARRISON FRENCH, elected Associate Member December 5th, 1906; died July 9th, 1909.

HENRY BROWN RICHARDSON, elected Member May 7th, 1879; died August 21st, 1909.

HENRY MAYNADIER STEELE, elected Associate Member April 5th, 1893; Member December 6th, 1899; died July 5th, 1909.

Adjourned.

OF THE BOARD OF DIRECTION

(Abstract)

August 31st, 1909.—Vice-President Pegram in the chair; Chas. Warren Hunt, Secretary, and present, also, Messrs. Christie, Kittredge, Knap, Thompson, and Williams.

Ballots for membership were canvassed resulting in the election of 15 Members, 20 Associate Members, 3 Associates, and 18 Juniors, and the transfer of 4 Associate Members, 1 Associate, and 1 Junior to the grade of Member, and 4 Juniors to the grade of Associate Member.

The report of the Nominating Committee was received.

The President was authorized to appoint a Committee of the Board to consider and report as to the time and place for holding the next Annual Convention.

The following resignations were accepted: Howard A. Dill, M. Am. Soc. C. E., C. W. Kinder, M. Am. Soc. C. E., John Young, M. Am. Soc. C. E., Charles H. Umstead, Assoc. M. Am. Soc. C. E., and Hobart B. Upjohn, Assoc. M. Am. Soc. C. E.

Applications were considered and other routine business transacted.

Adjourned.

ANNOUNCEMENTS

The House of the Society is open from 9 A. M. to 10 P. M., every day, except Sundays, Fourth of July, Thanksgiving Day, and Christmas Day.

MEETINGS

October 6th, 1909.—8.30 P.M.—At this meeting two papers will be presented for discussion, as follows: "A Review of Chicago Paving Practice," by P. E. Green, Assoc. M. Am. Soc. C. E., and "The Purification of the Water Supply of Steelton, Pennsylvania," by James H. Fuertes, M. Am. Soc. C. E.

These papers are printed in *Proceedings* for August, 1909.

October 20th, 1909.—8.30 P. M.—A paper by Chester W. Larner, Esq., on "Characteristics of Modern Hydraulic Turbines," will be presented for discussion.

This paper is printed in *Proceedings* for August, 1909.

November 3d, 1909.—8.30 P. M.—At this meeting a paper by T. Howard Barnes, M. Am. Soc. C. E., entitled "The Reinforced Concrete Wharf of the United Fruit Company at Bocas del Toro, Panama," will be presented for discussion.

This paper is printed in this number of *Proceedings.*

November 17th, 1909.—8.30 P. M.—A paper by F. E. Trask, M. Am. Soc. C. E., on "The Outlet Control of Little Bear Valley Reservoir," will be presented for discussion.

This paper is printed in this number of *Proceedings.*

The first two of a series of papers descriptive of the New York Tunnel Extension of the Pennsylvania Railroad are published in this number. The first, by Gen. Charles Walker Raymond, M. Am. Soc. C. E., is an introductory paper covering the entire work; the second, by Alfred Noble, Past-President, Am. Soc. C. E., describes in a general way the East River Division. It is expected that papers describing in more detail the "Cross-Town Tunnels" and the "East River Tunnels" will be published in the October number of *Proceedings,* and scheduled for discussion at subsequent meetings. Other papers, covering the Terminal Station, Sunnyside Yard, and the Plant used in driving the East River Tunnels, will be published soon. A series of papers covering the North River Division of this great work is also promised.

MEETINGS OF THE SAN FRANCISCO ASSOCIATION OF MEMBERS, AM. SOC. C. E.

The San Francisco Association of Members of the American Society of Civil Engineers holds regular bi-monthly meetings with banquet, and weekly informal luncheons. The former are held at

6 P. M. at the Fairmont Hotel, on the third Friday of February, April, June, August, October, and November, and also on the third Wednesday of December, the latter being the Annual Meeting of the Association.

Informal luncheons are held at 12.15 P. M. every Wednesday, and the place of meeting may be ascertained by communicating with the Secretary of the Association, E. T. Thurston, Jr., Assoc. M. Am. Soc. C. E., 623 Wells Fargo Building.

The by-laws of the Association provide for the extension of hospitality to any members of the Society who may be temporarily in San Francisco, and any such member will be gladly welcomed as a guest of the Association at any of the above meetings, if he will notify the Secretary that he is in San Francisco.

PAPERS AND DISCUSSIONS

The first and second volumes of *Transactions* for 1909 (Volumes LXII and LXIII) have been issued. The third volume will be issued on September 30th, and the fourth on December 31st.

It is hoped that members and others who take part in the discussion of the papers presented will revise their remarks promptly, and that all written communications from those who cannot attend the meetings will be sent in at the earliest possible date after the issue of a paper in *Proceedings*. The issue of volumes of *Transactions* is dependent on the closing of discussions, and the co-operation of the membership will now be more necessary in this matter than heretofore, because four volumes are to be issued during the year, instead of two, and, to accomplish this, a definite date of issue for each has been established.

All papers accepted by the Publication Committee are classified by the Committee with respect to their availability for discussion at meetings.

Papers, which, from their general nature, appear to be of a character suitable for oral discussion, will be published as heretofore in *Proceedings*, and set down for presentation to a future meeting of the Society, and, on these, oral discussion, as well as written communications, will be solicited.

All papers which do not come under this heading, that is to say, those which, from their mathematical or technical nature, in the opinion of the Committee, are not adapted to oral discussion, will not be scheduled for presentation to any meeting. Such papers will be published in *Proceedings* in the same manner as those which are to be presented at meetings, but written discussions, only, will be requested for subsequent publication in *Proceedings* and with the paper in the volumes of *Transactions*.

PRIVILEGES OF ENGINEERING SOCIETIES EXTENDED TO MEMBERS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

Members of the American Society of Civil Engineers will be welcomed by the following Engineering Societies, both to the use of their Reading Rooms and at all Meetings:

American Institute of Mining Engineers, 29 West Thirty-ninth Street, New York City.

Associação dos Engenheiros Civis Portuguezes, Lisbon, Portugal.

Australasian Institute of Mining Engineers, Melbourne, Victoria, Australia.

Boston Society of Civil Engineers, 715 Tremont Temple, Boston, Mass.

Brooklyn Engineers' Club, 197 Montague Street, Brooklyn, N. Y.

Canadian Society of Civil Engineers, 413 Dorchester Street, West, Montreal, Que., Canada.

Civil Engineers' Society of St. Paul, St. Paul, Minn.

Cleveland Engineering Society, 718 Caxton Building, Cleveland, Ohio.

Cleveland Institute of Engineers, Middlesbrough, England.

Colorado Association of Members, Am. Soc. C. E., H. J. Burt, Secy., 235 Equitable Building, Denver, Colo.

Engineers' and Architects' Club of Louisville, Ky., 303 Norton Building, Fourth and Jefferson Streets, Louisville, Ky.

Engineers' Club of Baltimore, Baltimore, Md.

Engineers' Club of Central Pennsylvania, Corner Second and Walnut Streets, Harrisburg, Pa.

Engineers' Club of Minneapolis, 17 South Sixth Street, Minneapolis, Minn.

Engineers' Club of Philadelphia, 1317 Spruce Street, Philadelphia, Pa.

Engineers' Club of St. Louis, 3817 Olive Street, St. Louis, Mo.

Engineers' Club of Toronto, 96 King Street, West, Toronto, Ont., Canada.

Engineers' Society of Western Pennsylvania, 803 Fulton Building, Pittsburg, Pa.

Institute of Marine Engineers, 58 Romford Road, Stratford, London, E., England.

Institution of Engineers of the River Plate, Buenos Aires, Argentine Republic.

Institution of Naval Architects, 5 Adelphi Terrace, London, W. C., England.

Junior Institution of Engineers, 39 Victoria Street, Westminster, S. W., London, England.

Koninklijk Instituut van Ingenieurs, The Hague, The Netherlands.
Louisiana Engineering Society, 604 Tulane-Newcomb Building, New Orleans, La.
Memphis Engineering Society, Memphis, Tenn.
Midland Institute of Mining, Civil and Mechanical Engineers, Sheffield, England.
Montana Society of Engineers, Butte, Montana.
North of England Institute of Mining and Mechanical Engineers, Newcastle-upon-Tyne, England.
Oesterreichischer Ingenieur- und Architekten-Verein, Eschenbachgasse 9, Vienna, Austria.
Pacific Northwest Society of Engineers, 617-618 Pioneer Building, Seattle, Wash.
Rochester Engineering Society, Rochester, N. Y.
Sachsischer Ingenieur- und Architekten-Verein, Dresden, Germany.
Sociedad Colombiana de Ingenieros, Bogota, Colombia.
Societe des Ingenieurs Civils de France, 19 Rue Blanche, Paris, France.
Society of Engineers, 17 Victoria Street, Westminster, S. W., London, England.
Svenska Teknologforeningen, Brunkebergstorg 18, Stockholm, Sweden.
Tekniske Forening, Vestre Boulevard 18–1, Copenhagen, Denmark.
Western Society of Engineers, 1737 Monadnock Block, Chicago, Ill.

SEARCHES IN THE LIBRARY

In January, 1902, the Secretary was authorized to make searches in the Library, upon request, and to charge therefor the actual cost to the Society for the extra work required. Since that time many searches have been made, and bibliographies and other information on special subjects furnished.

The resulting satisfaction, to the members, who have made use of the resources of the Society in this manner, has been expressed frequently, and leaves little doubt that, if it were generally known to the membership that such work would be undertaken, many would avail themselves of it.

The cost is trifling compared with the value of the time of an engineer who looks up such matters himself, and the work can be performed quite as well, and much more quickly, by persons familiar with the Library.

In asking that such work be undertaken, members should specify clearly the subject to be covered, and whether references to general books only are desired, or whether a complete bibliography, involving search through periodical literature, is desired.

In reference to this work, the Appendix* to the Annual Report of the Board of Direction for the year ending December 31st, 1906, contains a summary of all searches made to that date.

* *Proceedings*, Vol. XXXIII, p. 20 (January, 1907).

ACCESSIONS TO THE LIBRARY

(From August 2d to September 7th, 1909)

DONATIONS*

REINFORCED CONCRETE IN EUROPE.

By Albert Ladd Colby, M. Am. Soc. C. E. Cloth, 9 x 5¾ in., 10 + 260 pp. Easton, Pa., The Chemical Publishing Co.; London, Williams & Norgate, 1909. $3.50. (Donated by the Author.)

The author is authority for the statement that this report, which was originally issued in a private edition for subscribers, is a compilation of information on current practice in reinforced concrete construction in Great Britain and on the Continent. It was collected during 1908 chiefly by personal interviews with the leading authorities in each country. The theoretical branches of the subject, including the rules for calculation, are, he states, but briefly referred to, but the practical branches are fully treated. Addresses of prominent consulting and contracting engineers of each country are also given, in order to enable the reader to obtain further information if desired. The bibliography of books, journals, and periodicals of each country, contained in Appendix No. 4, gives the price and date of each publication. The Contents are: Applications of Reinforced Concrete; Economies of Reinforced Concrete Construction; Endurance of Foreign Reinforced Concrete Construction; Foreign Systems of Reinforced Concrete Construction; Mechanical Bond and Forms of Bars; Metal Used for Reinforcement, Foreign Specifications, Recommendations and Opinions Compared; Cement Used in Reinforced Concrete, The Chief Requirements of Foreign Cement Specifications Compared; Concrete Used in Reinforced Concrete. The Chief Requirements of Foreign Concrete Specifications Compared; Reinforced Concrete, Foreign Specifications and Recommendations Compared under the Chief Specified Requirements; Lists and Descriptions of Foreign Government and Private Testing Stations, Congresses, Technical Institutions, Associations, and Committees, Who Have Endorsed Reinforced Concrete as a Material or Who Have Adopted Resolutions, Specifications, or Rules Relating Thereto; Bibliography on Reinforced Concrete, Concrete and Cement; Index.

BRICKLAYING SYSTEM.

By Frank B. Gilbreth. Cloth, 9¼ x 6 in., illus., 11 + 321 pp. New York and Chicago, The Myron C. Clark Publishing Co.; London, E. & F. N. Spon, Ltd., 1909. $3.00 net.

The purpose of this work is declared by the author in his introduction to be threefold, first, to put in writing that knowledge which has been handed down by word of mouth from journeyman to apprentice for generations; second, to record methods of handling labor, materials, and plant on brickwork which will reduce costs and, at the same time, enable the first-class workman to receive better pay; third, to enable an apprentice to work intelligently from his first day, and to become a proficient workman in the shortest possible time. The Contents are: Training Apprentices; Methods of Management; Methods of Construction; Routing of Material; Scaffolds, The Gilbreth Scaffold-Hod Type; The Gilbreth Scaffold-Packet Type; The Gilbreth Packet System; Method of Building Tall Chimneys; Mortar; Bricks; Bricklayers' Tools, etc.; Lines, Plumbs and Poles; Motion Study; Methods of Laying Brick Under Special Conditions; Finishing, Jointing and Pointing; Arches and Chimney Breasts; Tearing Down, Cutting Out and Patching Brickwork; Bond; Glossary of Terms Used; Index.

IRRIGATION ENGINEERING.

By Herbert M. Wilson, M. Am. Soc. C. E. Sixth Edition, Revised and Enlarged. Cloth, 9 x 6 in., illus., 30 + 625 pp. New York, John Wiley & Sons; London, Chapman & Hall, Ltd., 1909. $4.00.

The preface of this edition, which is the fifth since the publication of the original work in 1893 under the title "Manual of Irrigation Engineering," states that it has been almost entirely rewritten, bringing up to date the tremendous progress made in construction by the Reclamation Service. Since the last edition, it is further stated, important changes have been produced in the design and in the structural materials used on irrigation works, as a result of the general adoption of reinforced concrete for such works. Every chapter and every article of this edition have undergone changes, including the elimination of much old matter and illustrations, and the introduction of new textual matter and eighty new illustrations representa-

*Unless otherwise specified, books in this list have been donated by the publisher.

tive of more modern designs. Many of the chapters conclude with bibliographies of their subjects. The Contents are: Introduction; Hydrography: Precipitation, Run-off, and Stream Flow; Evaporation, Absorption, and Seepage; Alkali, Drainage, and Sedimentation; Quantity of Water Required; Flow and Measurement of Water in Open Channels; Subsurface Water Sources and Sewage for Irrigation. Canals and Canal Works: Classes of Irrigation Works; Alignment, Slope, and Cross-Section; Headworks and Diversion Weirs; Sluiceways, Regulators, and Escapes; Falls and Drainage Works; Distributaries; Application of Water and Pipe Irrigation. Storage Reservoirs: Location and Capacity of Reservoirs; Earth and Loose-Rock Dams; Masonry Dams; Wasteways and Outlet Sluices; Pumping, Tools and Maintenance; Reclamation Service of the United States; Tables; Index.

MODERN METHODS OF STREET CLEANING.

By George A. Soper, M. Am. Soc. C. E. Cloth, 9 x 6 in., illus., 8 + 201 pp. New York, The Engineering News Publishing Company; London, Archibald Constable & Co., Ltd., 1909. $3.00.

These notes, the author declares in his foreword, are based, not only on his own experience, but more particularly on studies made during a three months' visit to Europe in 1907. Much of the statistical information contained in this book was, he states, supplied by officials. The Contents are: The Use and Abuse of City Streets; Fundamental Principles Applicable to Street Cleaning; The Cleaning of Pavements; The Disposal of Refuse; Summary of European Methods of Street Cleaning; London; Westminster, London; Manchester; Paris; Berlin; Hamburg; Cologne; Amsterdam; New York; Index.

SANITARY ENGINEERING.

A Practical Treatise on the Collection, Removal, and Final Disposal of Sewage and House Refuse and the Design and Construction of Works of Drainage and Sewerage, with Numerous Hydraulic Tables, Formulæ and Memoranda, Including an Extensive Series of Tables of Velocity and Discharge of Pipes and Sewers. By E. C. S. Moore. Third Edition, Revised and in Part Re-written by E. J. Silcock. Cloth, 9¼ x 6 in., illus., 2 v. Philadelphia, J. B. Lippincott Co., 1909. $14.00 net.

This revision of Moore's "Sanitary Engineering," the preface states, retains not only the original arrangement of the book, but follows, as far as possible, the same lines of treatment used by the late author. It is also stated that considerable alterations and additions to practically every chapter have been necessitated by the rapid advance of sanitary science, entire chapters on sewage purification have been re-written, and representative works embodying the latest and most approved practice have been described and illustrated. The Contents are: Introduction; Conservancy Systems; Sewerage; Sewage Lifting; Construction and Materials; The Flow of Liquid in Pipes and Open Channels; Hydraulic Memoranda and Tables; Application of Formulæ; Sewer and Drain Ventilation; Traps; Apparatus—Latrines and W. C.'s; Apparatus—Urinals, Lavatory Fitments, etc.; Surface Water Collection; Subsoil Drainage; Sanitary Notes (Drain Testing, Disinfection, etc.); Sewage Disposal—The Problem; Sewage Disposal—Methods of Treatment; Sewage Disposal—Artificial Bacterial Methods; Sewage Disposal—Sludge; Description of Existing Sewage Purification Works; Reports of the Royal Commission on Sewage Disposal and Requirements of the Local Government Board; Destructors; Construction of Chimney Shafts for Destructors; Appendix No. 1, Trade Effluents; Appendix No. 2, Sewage Fungus; Appendix No. 3, Local Government Requirements with Respect to Sewerage and Sewage Disposal (Revised 1909); List of Plates; List of Tables; List of Books Relating to the Subject; Addresses of Manufacturers Whose Specialties Are Referred To; Index.

SEWAGE PURIFICATION AND DISPOSAL.

By J. J. Cosgrove. Cloth, 8½ x 5½ in., illus., 8 + 222 pp. Pittsburg, Standard Sanitary Mfg. Co., 1909. $3.00.

The author of this work, in his preface, claims that up to the present time no textbook or treatise has been published which gives, in concise, ready form, rules, tables, and data for designing and proportioning sewage purification works, and, further, that the principles and data worked out by experiment and experience are scattered through such widely separated reports, public documents, and private papers, that they are not in available form. For these reasons, he states, he has prepared the present volume, presenting, as simply as possible, a work

which shows the method of constructing various types of sewage purification plants, their details and proportions, with a description of materials best suited to the purpose. To this end the illustrations are simple and are perspective instead of mechanical drawings, it being assumed by the author that the designer possesses the necessary skill as draftsman to make plans, and simply wants to know the size, shape, and materials of the several parts. The Contents are: Composition of Sewage; Decomposition of Sewage; The Septic Tank; Intermittent Filtration of Sewage; Sprinkling Filters; Contact Beds; Sewage Irrigation; Subsurface Irrigation; Chemical Precipitation; Sewage Pumping Plants; Disposal of Sewage; Disposal of Sludge; Disposal of Effluents; Disposal of Storm Water; Designing Sewage Purification Plants; Appendix (by Allen Hazen): Some Physical Properties of Sands and Gravels with Special Reference to Their Use in Filtration; Index.

THE SANITATION, WATER SUPPLY AND SEWAGE DISPOSAL OF COUNTRY HOUSES.

By Wm. Paul Gerhard. Cloth, 7¾ x 5 in., illus., 20 + 328 pp. New York, D. Van Nostrand Co., 1909. $2.00 net.

This book is divided into three parts. Part I, Sanitation, treats of the general sanitation of country houses. A condensed summary of the essential requirements of healthfulness is given, together with a brief discussion of the soil, subsoil, surface drainage, aspect, healthful surroundings and those which are objectionable, lighting, heating and ventilation, water supply, sewerage, and plumbing. In Part II, Water Supply, detailed advice is given as to how to obtain a satisfactory water supply, the sources of water, the various modes of raising it, its storage in reservoirs, elevated tanks, or underground pressure tanks, and its final distribution, being dwelt on at length and illustrated by actual examples. In Part III, Sewage Disposal, disposal for houses not in reach of sewers is discussed, particular attention being given, according to the author, to the latest developments in disposal methods, to the bacterial purification systems, including septic and cultivation tanks, contact filter beds, and sprinkling or trickling filters. The author states that, throughout the book, his aim has been to tell what to do rather than how to do it, and to establish sanitary principles which will lead to sanitary practice.

PRINCIPLES OF REINFORCED CONCRETE CONSTRUCTION.

By F. E. Turneaure, Assoc. M. Am. Soc. C. E., and E. R. Maurer. Second Edition, Revised and Enlarged. Cloth, 9 x 6 in., illus., 10 + 429 pp. New York, John Wiley & Sons; London, Chapman & Hall, Limited, 1909. $3.50.

The authors have endeavored, it is stated in the preface, to cover the principles of mechanics underlying the design of reinforced concrete, to present the results of all available tests that may aid in establishing coefficients and working stresses, and to illustrate from actual designs the principles involved. The book is divided into two parts, namely, Chapters I to VI which treat of the theory of the subject and the results of experiments, and Chapters VII to XI which treat of the use of reinforced concrete in various forms of structures. In the preface to this, the second edition, it is stated that numerous changes and additions to the various chapters have been made and a chapter on chimneys has been included, and also that new experimental data are given in nearly every division of the subject, those of especial importance being the results on bond strength, strength of beams in shear, and the strength of columns. The Contents are: Introductory; Properties of the Material; General Theory; Tests of Beams and Columns; Working Stresses and General Constructive Details; Formulas, Diagrams and Tables; Building Construction; Arches; Retaining Walls and Dams; Miscellaneous Structures; Reinforced-Concrete Chimneys; Index.

Gifts have also been received from the following:

Alexandra (Newport & South Wales) Docks & Ry. Co. 1 pam.
Am. Public Health Assoc. 3 pam.
Am. Soc. Mech. Engrs. 1 bound vol.
Baker St. & Waterloo Ry. Co. 1 pam.
Baltimore, Md.-Water Board. 1 bound vol.
Boston, Mass.-City Auditor. 1 bound vol.
Boston, Mass.-Mayor. 1 bound vol.
Buffalo, Rochester & Pittsburgh Ry. Co. 1 pam.
Bush, Lincoln. 4 pam.
Caird, J. M. 1 pam.
Cal.-State Mineralogist. 2 bound vol., 3 pam.
Cambrian Rys. Co. 1 pam.
Cambridge, Mass.-Water Board. 1 pam.
Canada-Mines Branch, Dept. of Mines. 1 pam.
Carnegie Library of Pittsburgh. 1 pam.
Central London Ry. Co. 1 pam.
Central of Georgia Ry. Co. 1 pam.
Charing Cross, Euston & Hampstead Ry. Co. 1 pam.
Earl, George G. 1 bound vol.

City Club of New York. 1 bound vol.
Dixon, Samuel G. 2 pam.
Egypt-Gen. Mgr. of State Rys. and Telegraphs. 1 bound vol.
Engrs.' Club. 1 bound vol.
Engrs.' Club of Philadelphia. 1 bound vol.
Fisk & Robinson. 2 pam.
Furness Ry. Co. 2 pam.
Great Britain-Home Office. 35 pam.
Great Central Ry. Co. 2 pam.
Great Eastern Ry. Co. 1 pam.
Great Northern & City Ry. Co. 1 pam.
Great Northern, Piccadilly & Brompton Ry. Co. 1 pam.
Great Northern Ry. Co. (Ireland). 1 pam.
Great Western Ry. 1 pam.
Hull & Barnsley Ry. Co. 1 pam.
India-Ry. Board. 1 vol.
Inst. of Marine Engrs. 1 vol.
Institution of Civ. Engrs. 1 bound vol.
Iowa Eng. Soc. 1 pam.
Iron and Steel Inst. 1 bound vol., 1 pam.
Koester, Frank. 2 bound vol.
Kgl. Technische Hochschule zu Berlin. 1 pam.
Leisen, Theodore A. 1 pam.
Liverpool Overhead Ry. Co. 1 pam.
London, Brighton & South Coast Ry. Co. 1 pam.
London, Tilbury & Southend Ry. Co. 1 pam.
Louisville Water Co. 1 pam.
McGill Univ. 1 bound vol.
Madras, India-Public Works Dept. 1 bound vol.
Magyar Mernök-és Epítész-Egylet. 1 pam.
Mailloux, C. O. 1 pam.
Mass.-Agri. Exper. Station. 1 pam.
Merchant's Assoc. of New York. 1 pam.
Met. Ry. Co. 1 pam.
Mich. Coll. of Mines. 1 pam.
Midland & South Western Junction Ry. 1 pam.
Midland Ry. Co. 1 pam.
New England Water Works Assoc. 1 pam.
New South Wales-Govt. Board for Inter. Exchanges. 3 bound vol., 7 pam.
New South Wales-Met. Board of Water Supply and Sewerage. 1 vol.
New York City-Board of *City Record*. 5 bound vol.
New York City-Met. Sewerage Comm. 1 pam.
New York State-Public Service Comm. for the First Dist. 1 pam.
North, Edward P. 2 vol.
North London Ry. Co. 1 pam.
North Staffordshire Ry. Co. 1 pam.
Oklahoma-Corporation Comm. 1 bound vol.
Ontario, Canada-Bureau of Mines. 1 pam.
Pennsylvania-Commr. of Health. 1 bound vol.
Pittsburg, Pa., Chamber of Commerce. 1 pam.
Rensselaer Polytechnic Inst. 1 pam.
Rhymney Ry. 1 pam.
Rose Polytechnic Inst. 1 bound vol.
Rust, Charles H. 1 bound vol.
Ruttan, Henry N. 2 pam.
Saginaw, Mich.-Board of Water Commrs. 1 pam.
St. John, N. B.-City Engr. 1 pam.
Soc. for the Promotion of Eng. Education. 1 bound vol.
South Eastern Ry. Co. 1 pam.
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U. S.-Bureau of Education. 1 pam.
U. S.-Bureau of the Census. 1 bound vol., 1 pam.
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U. S.-Lake Survey Office. 1 pam.
U. S.-National Museum. 1 vol.
U. S.-Office of Library and Naval War Records. 1 vol.
Univ. of Georgia. 1 vol.
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Updegraff, Milton. 2 bound vol.
Verein Deutscher Ingenieure. 1 vol.
Victoria, Australia-Sludge Abatement Board. 1 pam.
Victorian Inst. of Engrs. 1 bound vol.
Vienna, Austria-Magistrats. 1 bound vol.
Western Soc. of Engrs. 1 pam.

BY PURCHASE

Rapid Methods for the Chemical Analysis of Special Steels, Steelmaking Alloys and Graphite. By Charles Morris Johnson. John Wiley & Sons, New York; Chapman & Hall, Ltd., London, 1909.

Index of Mining Engineering Literature, Comprising an Index of Mining, Metallurgical, Civil, Mechanical, Electrical, and Chemical Engineering Subjects as Related to Mining Engineering. By Walter R. Crane. John Wiley & Sons, New York; Chapman & Hall, Ltd., London, 1909.

History of the Clay-Working Industry in the United States. By Heinrich Ries and Henry Leighton. John Wiley & Sons, New York; Chapman & Hall, Ltd., London, 1909.

Tables and Diagrams of the Thermal Properties of Saturated and Superheated Steam. By Lionel S. Marks and Harvey N. Davis. Longmans, Green & Co., New York, London, Bombay and Calcutta, 1909.

The Panama Canal and Its Makers. By Vaughan Cornish. Little, Brown & Co., Boston, 1909.

Electro-Magnetic Ore Separation. By C. Godfrey Gunther. Hill Publishing Co., New York, 1909.

SUMMARY OF ACCESSIONS

(From August 2d to September 7th, 1909)

Donations (including 5 duplicates)	173
By purchase	6
Total	179

MEMBERSHIP

ADDITIONS

(August 2d to September 7th, 1909)

	Date of Membership.
MEMBERS	
ECKERSLEY, JOSEPH OSCAR. 4269 White Plains Ave., New York City	Aug. 31, 1909
KIBBE, AUGUSTUS SAYRE. Chf. Engr., The Am. Rys. Co., 906 Witherspoon Bldg., Philadelphia, Pa.	Jun. Mar. 6, 1889 M. Aug. 31, 1909
TINKER, GEORGE HENRY. 420 Hickox Bldg., Cleveland, Ohio	Aug. 31, 1909
ASSOCIATE MEMBERS	
CARSTARPHEN, FREDERICK CHARLES. 517 Ideal Bldg., Denver, Colo	April 6, 1909
CORLETT, BERTRAM EDWIN. 602 D. S. Johnston Co. Bldg., Seattle, Wash	May 4, 1909
DERBY, CHESTER CAWTHORNE. Chf. Draftsman, Office of Insp. of Public Works, 50 Elm St., Richmond Hill, N. Y	May 4, 1909
FREEMAN, MILTON HARVEY. 135 East 63d St., New York City	July 1, 1909
GRANT, ULYSSES S., 3D. 1st Lieut., Corps of Engrs., U. S. A., State, War and Navy Bldg., Washington, D. C	Aug. 31, 1909
HAYES, RALPH DANIEL. Stillwater, N. Y	Jun. June 4, 1907 Assoc. M. Aug. 31, 1909
MERRILL, OSCAR CHARLES. First National Bank Bldg., San Francisco, Cal	April 6, 1909
MILLER, STANLEY ALFRED. Hydr. Engr., Azua, Santo Domingo	Jun. Feb. 4, 1902 Assoc. M. April 6, 1909
NEWTON, SAMUEL DONALD. City Engr., Knoxville, Tenn	June 1, 1909
PERLEY, ALAN BIGELOW. 46 Gowen Ave., Mt. Airy, Philadelphia, Pa	June 1, 1909
REEVES, CARL HOWELL. Asst. Civ. Engr., Seattle Elec. Co., 5247 Fourteenth Ave., N. E., Seattle, Wash	Jun. May 2, 1905 Assoc. M. July 1, 1909
RICHARDSON, REX DENSMORE. 1035 Woodlawn, Scranton, Pa	Aug. 31, 1909
ROCKWELL, EDWARD HENRY. Prof. of Structural Eng., Tufts College, Tufts College, Mass	June 1, 1909
SHOEMAKER, HARRY IVES. Puunene, Maui, Hawaii	May 4, 1909
SOUTHWORTH, EDWARD AUGUSTUS. Dept. of Public Works, Honolulu, Hawaii	July 1, 1909
TILMONT, PAUL ALPHONSE GAILLARD. Res. Engr., N. P. Ry., Nimrod, Mont	July 1, 1909
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CUNNINGHAM, PINKNEY EDWARD. P. O. Box 404, Vicksburg, Miss.....	July 1, 1909
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FORSYTH, HAROLD FREDERICK. Lone Tree P. O., Chehalis Co., Wash.....	Mar. 2, 1909
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LICHTNER, WILLIAM OTTO. Newton Highlands, Mass.....	Aug. 31, 1909
MACKALL, JOHN NATHANIEL. Md. Geol. Survey, Baltimore, Md.....	Aug. 31, 1909
SMITH, SHALER GORDON. Care, Interborough Rapid Transit Co., 32 Park Pl., New York City.....	July 1, 1909
TEAL, JONATHAN ERNEST. Asst. Engr., Mo. Pac. Ry. Co., St. Louis, Mo.....	June 1, 1909
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WILSON, WILLIAM WEST. Elda Farm, Berwyn, Pa.
WOOD, ROBERT WALTER. Asst. Engr., in Chg. of Port Richmond Div., Borough of Richmond, New York City; Address, 13 Van Pelt Ave., Mariner's Harbor, N. Y.

RESIGNATIONS

MEMBERS	Date of Resignation
DILL, HOWARD ALBERT	Aug. 31, 1909
KINDER, CLAUDE WILLIAM	Aug. 31, 1909
YOUNG, JOHN	Aug. 31, 1909
ASSOCIATE MEMBERS	
UMSTEAD, CHARLES HENRY	Aug. 31, 1909
UPJOHN, HOBART BROWN	Aug. 31, 1909

DEATHS

BISHOP, GEORGE H. Elected Member, August 7th, 1872; died August 20th, 1909.
MORTON, CHARLES. Elected Member, October 2d, 1895; died August 28th, 1909.
RICHARDSON, HENRY BROWN. Elected Member, May 7th, 1879; died August 21st, 1909.

Total Membership of the Society, September 14th, 1909

5119

MONTHLY LIST OF RECENT ENGINEERING ARTICLES OF INTEREST

(August 2d to September 7th, 1909)

NOTE.—*This list is published for the purpose of placing before the members of the Society, the titles of current engineering articles, which can be referred to in any available engineering library, or can be procured by addressing the publication directly, the address and price being given wherever possible.*

LIST OF PUBLICATIONS

In the subjoined list of articles, references are given by the number prefixed to each journal in this list:

(1) *Journal,* Assoc. Eng. Soc., 31 Milk St., Boston, Mass., 30c.
(2) *Proceedings,* Engrs. Club of Phila., 1317 Spruce St., Philadelphia, Pa.
(3) *Journal,* Franklin Inst., Philadelphia, Pa., 50c.
(4) *Journal,* Western Soc. of Engrs., Monadnock Blk., Chicago, Ill.
(5) *Transactions,* Can. Soc. C. E., Montreal, Que., Canada.
(6) *School of Mines Quarterly,* Columbia Univ., New York City, 50c.
(7) *Technology Quarterly,* Mass. Inst. Tech., Boston, Mass., 75c.
(8) *Stevens Institute Indicator,* Stevens Inst., Hoboken, N. J., 50c.
(9) *Engineering Magazine,* New York City, 25c.
(10) *Cassier's Magazine,* New York City, 25c.
(11) *Engineering* (London), W. H. Wiley, New York City, 25c.
(12) *The Engineer* (London), International News Co., New York City, 35c.
(13) *Engineering News,* New York City, 15c.
(14) *The Engineering Record,* New York City, 12c.
(15) *Railroad Age Gazette,* New York City, 15c.
(16) *Engineering and Mining Journal,* New York City, 15c.
(17) *Electric Railway Journal,* New York City, 10c.
(18) *Railway and Engineering Review,* Chicago, Ill., 10c.
(19) *Scientific American Supplement,* New York City, 10c.
(20) *Iron Age,* New York City, 10c.
(21) *Railway Engineer,* London, England, 25c.
(22) *Iron and Coal Trades Review,* London, England, 25c.
(23) *Bulletin,* American Iron and Steel Assoc., Philadelphia, Pa.
(24) *American Gas Light Journal,* New York City, 10c.
(25) *American Engineer,* New York City, 20c.
(26) *Electrical Review,* London, England.
(27) *Electrical World,* New York City, 10c.
(28) *Journal,* New England Water-Works Assoc., Boston, Mass., $1.
(29) *Journal,* Royal Society of Arts, London, England, 15c.
(30) *Annales des Travaux Publics de Belgique,* Brussels, Belgium.
(31) *Annales* de l'Assoc. des Ing. Sortis des Ecoles Spéciales de Gand, Brussels, Belgium.
(32) *Mémoires et Compte Rendu des Travaux,* Soc. Ing. Civ. de France, Paris, France.
(33) *Le Génie Civil,* Paris, France.
(34) *Portefeuille Economiques des Machines,* Paris, France.
(35) *Nouvelles Annales de la Construction,* Paris, France.
(37) *Revue de Mécanique,* Paris, France.
(38) *Revue Générale des Chemins de Fer et des Tramways,* Paris, France.
(41) *Modern Machinery,* Chicago, Ill., 10c.
(42) *Proceedings,* Am. Inst. Elec. Engrs., New York City, 50c.
(43) *Annales des Ponts et Chaussées,* Paris, France.
(44) *Journal,* Military Service Institution, Governors Island, New York Harbor, 50c.
(45) *Mines and Minerals,* Scranton, Pa., 20c.
(46) *Scientific American,* New York City, 8c.
(47) *Mechanical Engineer,* Manchester, England.
(48) *Zeitschrift,* Verein Deutscher Ingenieure, Berlin, Germany.
(49) *Zeitschrift für Bauwesen,* Berlin, Germany.
(50) *Stahl und Eisen,* Düsseldorf, Germany.
(51) *Deutsche Bauzeitung,* Berlin, Germany.
(52) *Rigasche Industrie-Zeitung,* Riga, Russia.
(53) *Zeitschrift,* Oesterreichischer Ingenieur und Architekten Verein, Vienna, Austria.
(54) *Transactions,* Am. Soc. C. E., New York City, $5.
(55) *Transactions,* Am. Soc. M. E., New York City, $10.
(56) *Transactions,* Am. Inst. Min. Engrs., New York City, $5.

(57) *Colliery Guardian*, London, England.
(58) *Proceedings*, Eng. Soc. W. Pa., 803 Fulton Bldg., Pittsburg, Pa., 50c.
(59) *Transactions*, Mining Inst. of Scotland, London and Newcastle-upon-Tyne, England.
(60) *Municipal Engineering*, Indianapolis, Ind., 25c.
(61) *Proceedings*, Western Railway Club, 225 Dearborn St., Chicago, Ill., 25c.
(62) *Industrial World*, 59 Ninth St., Pittsburg, Pa.
(63) *Minutes of Proceedings*, Inst. C. E., London, England.
(64) *Power*, New York City, 20c.
(65) *Official Proceedings*, New York Railroad Club, Brooklyn, N. Y., 15c.
(66) *Journal of Gas Lighting*, London, England, 15c.
(67) *Cement and Engineering News*, Chicago, Ill., 25c.
(68) *Mining Journal*, London, England.
(70) *Engineering Review*, New York City, 10c.
(71) *Journal*, Iron and Steel Inst., London, England.
(73) *Electrician*, London, England, 18c.
(74) *Transactions*, Inst. of Min. and Metal., London, England.
(75) *Proceedings*, Inst. of Mech. Engrs., London, England.
(76) *Brick*, Chicago, Ill., 10c.
(77) *Journal*, Inst. Elec. Engrs., London, England.
(78) *Beton und Eisen*, Vienna, Austria.
(79) *Forscherarbeiten*, Vienna, Austria.
(80) *Tonindustrie Zeitung*, Berlin, Germany.
(81) *Zeitschrift für Architektur und Ingenieurwesen*, Wiesbaden, Germany.
(82) *Dinglers Polytechnisches Journal*, Berlin, Germany.
(83) *Progressive Age*, New York City, 15c.
(84) *Le Ciment*, Paris, France.
(85) *Proceedings*, Am. Ry. Eng. and M. of W. Assoc., Chicago, Ill.
(86) *Engineering-Contracting*, Chicago, Ill., 10c.
(87) *Roadmaster and Foreman*, Chicago, Ill., 10c.
(88) *Bulletin* of the International Ry. Congress Assoc., Brussels, Belgium.
(89) *Proceedings*, Am. Soc. for Testing Materials, Philadelphia, Pa.
(90) *Transactions*, Inst. of Naval Archts., London, England.
(91) *Transactions*, Soc. Naval Archts. and Marine Engrs., New York City.
(92) *Bulletin*, Soc. d'Encouragement pour l'Industrie Nationale, Paris, France.
(93) *Revue de Métallurgie*, Paris, France, 4 fr. 50.
(94) *The Boiler Maker*, New York City, 10c.
(95) *International Marine Engineering*, New York City, 20c.
(96) *Canadian Engineer*, Toronto, Canada, 15c.
(97) *Turbine*, Berlin, Germany, 1 Mark.

LIST OF ARTICLES

Bridge.

Nickel-Steel Eye-Bars for Blackwell's Island Bridge.* William R. Webster, M. Am. Soc. C. E. (54) Vol. 64.
Distribution of Locomotive Axle-Loads, in Relation to Bridges.* James Dudley Ward Ball, Assoc. M. Inst. C. E. (63) Vol. 176.
Derrick Cars and Bridge Erection, with some Tests of Iron Pulley Blocks.* J. H. Prior. (4) Aug.
Method and Cost of Constructing a Reinforced Concrete Arch Street Bridge at Baltimore, Md. (86) Aug. 4.
A 259-Ft. Concrete Arch Bridge in Switzerland.* (13) Aug. 5.
The Construction of the Edmondson Avenue Bridge, Baltimore.* (14) Aug. 14.
An English Double-Deck Swing Bridge for Railway and Roadway Traffic.* (13) Aug. 19.
Blackwell's Island Bridge, New York.* Frank W. Skinner, M. Am. Soc. C. E. (11) Serial beginning Aug. 20.
Erection of Additional Girders, Williamsburg Bridge Towers.* (14) Aug. 21.
The Bridges of New York City.* T. Kennard Thomson. (9) Sept.
The Monroe St. Bridge, Spokane, Washington; a Concrete Bridge Containing a 281-ft. Arch.* (13) Sept. 2.
Construction et Montage du Viaduc des Fades.* L. Le Chatelier. (92) July.
Le Pont de l'Institut.* (84) July.
Etude Economique de l'Emploi de l'Acier au Carbone à grande Résistance, pour la Construction des Ponts.* J. A. L. Waddell. (33) Aug. 7.
Le Viaduc de l'Assopos (Grèce).* Georges Bodin. (33) Aug. 14.

Electrical.

A Hydro-Electric Installation in Nyassa-Land.* Alexander Spence Chalmers, Assoc. M. Inst. C. E. (63) Vol. 176.
The New Telefunken Method of Wireless Telegraphy. W. H. Eccles. (73) July 30.
Modern Submarine Telegraphy.* S. G. Brown. (Abstract of paper read before the Royal Inst.) (73) Serial beginning July 30.
The Corrosion of Metals Underground by Electrolysis.* A. A. Knudson. (3) Aug.

*Illustrated.

Electrical—(Continued.)

Losses, Induced Volts and Amperes in Armor and Lead Cover of Cables. H. W. Fisher. (42) Aug.
Pole-Face Losses.* Comfort A. Adams, A. C. Lanier, C. C. Pope and C. O. Schooley. (42) Aug.
Dimensions of Single-Phase Core and Shell Transformers, with Copper and Iron Circuits of Uniform Rectangular Cross-Section.* A. R. Low. (77) Aug.
The Dielectric Strength of Compressed Air.* E. A. Watson. (77) Aug.
The Kapp-Hopkinson Test on a Single Direct-Current Machine.* W. Lulofs. (77) Aug.
Flywheel Load Equalisers.* J. S. Peck. (77) Aug.
Central-Station Development at Lawrenceville, Ill.* (27) Aug. 5.
The Selection of Towers for High-Tension Transmission Lines.* N. J. Neall. (27) Aug. 5.
Modern Methods of Artificial Illumination.* Leon Gaster. (29) Serial beginning Aug. 6.
Triple-Frequency Currents in Neutrals of Three-Phase Star Connected Alternators.* E. W. Marchant and J. K. Catterson-Smith. (73) Aug. 6.
Electric Circuit Problems, Design and Testing. (57) Serial beginning Aug. 6.
The Transmission and Distribution Systems of the Rio de Janeiro Tramway, Light & Power Company.* L. L. Perry. (27) Aug. 12.
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Connections for a Single-Phase Wattmeter on a Three-Phase, Three-Wire System.* F. Conrad. (27) Aug. 19.
Types of Alternating-Current Commutator Motors and the Best Frequency for Railways.* F. Eichberg. (Abstract from *Elek. Zeit.*) (73) Aug. 20.
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The Weiss Laboratory Electromagnet.* M. P. Misslin. (27) Aug. 26.
The Distribution of Dielectric Stress in Three-Phase Cables.* W. M. Thornton and O. J. Williams. (Paper read before the Brit. Assoc.) (11) Aug. 27.
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La Fabrication de l'Acide Azotique et des Azotates au Moyen de l'Azote Atmosphérique (procédé Schönherr).* E. Lemaire. (33) July 24.

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The Combination of Reciprocating and Turbine Engines.* W. McK. Wisnom. (Paper read before the Inst. of Engrs. and Shipbuilders.) (11) Aug. 6.
Trials and Performances of the S. S. *Otaki*, Fitted with a Combination of Reciprocating and Turbine Machinery.* W. McK. Wisnom. (Paper read before the Inst. of Engrs. and Shipbuilders in Scotland.) (47) Serial beginning Aug. 13.
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An Investigation into the Heat-Losses in an Electric Power-Station.* Frank Huntingdon Corson, Assoc. M. Inst. C. E. (63) Vol. 176.
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Interchangeability and Methods of Securing it in Screw-Threads.* H. F. Donaldson. (75) Feb.
Test of Gas Compressor No. 3 of the New Orleans Gas Light Company, Saturday, January 9, 1909.* W. B. Gregory. (Paper read before the La. Eng. Soc.) (1) July.
The Calculation of Vibration and Whirling Speeds. Arthur Morley. (12) Serial beginning July 30.
Testing Machine for Applying Bending and Twisting Moments Simultaneously.* E. G. Coker. (From *Proc.*, Physical Soc. of Lond.) (47) July 30.
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Installation of Carburetted Water-Gas Plant at Aberdeen.* Samuel Milne. (Paper read before the North Brit. Assoc. of Gas Mgrs.) (66) Aug. 3.
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Influence of the Treatment of the Solubility of Steel in Sulphuric Acid, and Deductions Therefrom as to the Nature of the Treatment to Which the Material has been Previously Subjected; Researches of the Royal Institute for Testing Materials at Gross-Lichterfelde.* E. Heyn and O. Bauer. (71) Vol. 79.
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Prospecting Anthracite Mines by Drill Holes.* Frank Lynde. **(16)** Aug. 7.
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The Introduction of Electric Traction on Main-Line Railways. E. Frischmuth. (From *Elek. Kraftbetriebe u. Bahnen.*) (88) Aug.
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Nouvelle Locomotive Compound à Trois Essieux Couplés et à Roues Motrices de 1m, 75 de la Compagnie du Chemin de Fer du Nord.* Du Bousquet. **(38)** Aug.
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An Aerating and Flushing System for Percolating Sewage Filters.* Wm. Gavin Taylor, M. Am. Soc. C. E. **(13)** Aug. 19.
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The Bonding of New to Old Concrete.* E. P. Goodrich, M. Am. Soc. C. E. **(54)** Vol. 64.
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Difficulties in the Practical Work of Creosoting Timber. John C. Oakes. (*Professional Memoirs,* U. S. Engr. Bureau.) **(13)** Sept. 2.
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Progress on the Ashokan Reservoir.* **(14)** Sept. 4.
The Goderich Sedimentation Basin.* J. Grant MacGregor, Assoc. M. Am. Soc. C. E. **(14)** Sept. 4.
Etablissement des Canalisations d'Eau et de Gaz au Point de Vue des Intérêts Financiers et Sanitaires.* J. Gilbert. **(32)** June.
La Déferrisation des Eaux Souterraines.* Bousquet. **(35)** Aug.

Waterway.

Famagusta Harbour, Cyprus.* George Hobbs. **(63)** Vol. 176.
The Panama Canal.* Frederic P. Stearns. (Paper read before the Bost. Soc. of Civ. Engrs.) **(1)** July.
Concrete-Steel Caissons; Their Development and Use for Breakwaters, Piers and Revetments.* W. V. Judson, M. Am. Soc. C. E. **(4)** Aug.
Methods Employed in Computing Reinforced Concrete Caissons for Algoma Breakwater.* **(86)** Aug. 18.
Three Subways under Sydney Harbor, New South Wales.* C. O. Burge. **(14)** Aug. 21.
Rock Removal in Hell Gate, New York City.* **(14)** Aug. 21.
Recent Revetment Construction on the Lower Mississippi River.* Clarke S. Smith. **(13)** Aug. 26.
Memoranda on Dredging in Ambrose Channel, New York Harbor.* H. L. Wigmore. (From *Professional Memoirs*, U. S. Engr. Bureau.) **(13)** Aug. 26.
The Present Aspects of the Panama Canal from the Tourist's Point of View.* William Tatham. **(3)** Sept.
The Hydraulic Transmission of Dredged Materials at San Pedro Harbor, Cal.; a Record of Performance.* H. Hawgood, M. Am. Soc. C. E. **(13)** Sept. 2.
Redressement de l'Escaut en Aval d'Anvers. C.-J. Van Mierlo. **(31)** Pt. 2.
Les Echelles Hydrauliques Appliquées à la Canalisation et à la Régularisation des Rivières.* G. Denil. **(31)** Pt. 2.

*Illustrated.

PROCEEDINGS

OF THE

AMERICAN SOCIETY

OF

CIVIL ENGINEERS

(INSTITUTED 1852)

VOL. XXXV—No. 8.

OCTOBER, 1909.

Edited by the Secretary, under the direction of the Committee on Publications.

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NEW YORK 1909

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American Society of Civil Engineers

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ON STEEL COLUMNS AND STRUTS: Austin L. Bowman, Alfred P. Boller, Emil Gerber, Charles F. Loweth, Ralph Modjeski, Frank C. Osborn, George H. Pegram, Lewis D. Rights, George F. Swain, Emil Swensson, Joseph R. Worcester.

ON BITUMINOUS MATERIALS FOR ROAD CONSTRUCTION: W. W. Crosby, A. W. Dean, H. K. Bishop, A. H. Blanchard.

The House of the Society is open from 9 A.M. to 10 P.M. every day, except Sundays Fourth of July, Thanksgiving Day and Christmas Day.

HOUSE OF THE SOCIETY—220 WEST FIFTY-SEVENTH STREET, NEW YORK.

TELEPHONE NUMBER.....5913 Columbus.
CABLE ADDRESS....."Ceas, New York."

Vol. XXXV. OCTOBER, 1909. No. 8.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PROCEEDINGS

This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

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MINUTES OF MEETINGS OF THE SOCIETY

September 15th, 1909.—The meeting was called to order at 8.30 P. M.; James Owen, M. Am. Soc. C. E., in the chair; Chas. Warren Hunt, Secretary, and present, also, 124 members and 30 guests.

A paper by John C. Hoyt, M. Am. Soc. C. E., entitled "The Use and Care of the Current Meter, as Practiced by the United States Geological Survey," was presented by H. K. Barrows, M. Am. Soc. C. E., and discussed by Messrs. C. E. Grunsky, E. Kuichling, and H. K. Barrows. The Secretary read a discussion by A. H. Diamant, Assoc. M. Am. Soc. C. E.

A paper by George S. Webster and Henry H. Quimby, Members, Am. Soc. C. E., entitled "Walnut Lane Bridge, Philadelphia," was

presented by Mr. Quimby and illustrated with lantern slides. The paper was discussed by Messrs. B. C. Collier and Henry H. Quimby.

The Secretary announced that he had received the following telegram from Robert E. Peary, M. Am. Soc. C. E.:

"Glad inform you Member Society has attained North Pole."

Adjourned.

October 6th, 1909.—The meeting was called to order at 8.30 P. M.; President Onward Bates in the chair; Chas. Warren Hunt, Secretary; and present, also, 130 members and 25 guests.

The minutes of the meeting of September 1st, 1909, were approved as printed in *Proceedings* for September, 1909.

A paper by P. E. Green, Assoc. M. Am. Soc. C. E., entitled "A Review of Chicago Paving Practice," was presented by title. The Secretary read a communication on the subject by W. W. Crosby, M. Am. Soc. C. E., and the paper was discussed by Messrs. Richard Lamb, N. P. Lewis, and E. H. Thomes.

A paper by James H. Fuertes, M. Am. Soc. C. E., entitled "The Purification of the Water Supply of Steelton, Pa.," was presented by the author and illustrated with lantern slides. The paper was discussed by Mr. James E. Little, and the Secretary read communications on the subject by Messrs. H. V. Hinckley and William R. Copeland.

Richard Lamb, M. Am. Soc. C. E., moved that the Board of Direction be requested to consider the appointment of a Special Committee to investigate and report on the best complete specifications for wood block paving.

The motion was duly seconded, but did not receive the necessary affirmative vote of twenty-five Corporate Members and was declared lost.

The Secretary announced the election of the following candidates by the Board of Direction on October 5th, 1909:

As Members.

John Phillips Badenhausen, Seattle, Wash.
Roy Hamilton Beattie, Fall River, Mass.
Bernard Arthur Behrend, Pittsburg, Pa.
Harry Burgess, New Orleans, La.
George Linden Campen, Ancon, Canal Zone, Panama.
Clarence Knight Conard, New York City.
Henry Welles Durham, Sandwich, Mass.
John Witmer Emig, Denver, Colo.
Edward Godfrey, Pittsburg, Pa.
John Wesley Hackney, Atlantic City, N. J.
Heinrich Homberger, San Francisco, Cal.

Stephen Upshur Hopkins, Portland, Ore.
Chiranji Lall, Lahore, Punjab, India.
Edgar Heisler Lawrence, St. Louis, Mo.
Maurice Alvin Long, Baltimore, Md.
Robert Emmett McDonnell, Kansas City, Mo.
John Gilmore McMillan, San José, Cal.
Frederic Emery Pierce, New York City.
Frederick Longfellow Pitman, Spokane, Wash.
Charles Henry Preston, Norwich, Conn.
Adolph Eugene Schneeweiss, Paterson, N. J.
Arthur Ludwig Sonderegger, Los Angeles, Cal.
Leonard Lee Street, Dorchester, Mass.
Raymond F. Walter, Belle Fourche, S. Dak.
Carlos Wauters, Buenos Aires, Argentine Republic.
Charles Lawson Wooldridge, Pittsburg, Pa.
Aaron Stanton Zinn, Empire, Canal Zone, Panama.

As Associate Members.

Leland Rella Balch, Family, Mont.
Carroll Beale, Washington, D. C.
Harry Orlando Beale, Family, Mont.
Gurdon Gilmore Black, St. Louis, Mo.
Max Blatt, New York City.
Charles Frederick Blood, Ridgefield, N. J.
Thomas Franklin Boltz, Dillsburg, Pa.
Jesse Irving Brewer, New York City.
Burtis Scott Brown, Boston, Mass.
Alfred Dickey Butler, Spokane, Wash.
Samuel Dexter Clinton, Downey, Idaho.
Hewitt Crosby, Brooklyn, N. Y.
Louis Henry Egan, Rochester, Mich.
William Warrick Fineren, Tampa, Fla.
Jacques André Fouilhoux, Portland, Ore.
Archibald John Russell Frew, Almaden, Queensland, Australia.
Louis Goodman, New York City.
Edgar Miller Graham, Yonkers, N. Y.
Walter Stanley Haldeman, Philadelphia, Pa.
William Albert Hansell, Fort Meade, S. Dak.
Harry Hall Holley, Visalia, Cal.
Alfred Stowe Mirick, Albany, N. Y.
Lester Hale Mitchell, Glendive, Mont.
Ernest Montgomery, Palisades, Colo.
Theodore Parker Moorehead, Atlanta, Ga.
Charles Edward Morrison, New York City.
Edward Farnum Rockwood, Boston, Mass.

Cornelius Bramhall Sampson, Conejos, Colo.
Paul Sheldon, Hartford, Conn.
John Jenks Shonk, Bethlehem, Pa.
Charles Spearman, Boise, Idaho.
Ralph Raymond Wiggins, Gatun, Canal Zone, Panama.
Samuel Lamson Wonson, City of Mexico, Mexico.

As Juniors.

Claude Osgood Brown, Manila, Philippine Islands.
Amasa Manton Chace, Newport, R. I.
Frank Edward Dennie, Campello, Mass.
Lloyd Tilghman Emory, McIntyre, Ga.
Hudson Bridge Hastings, Brunswick, Me.
John Strider Hess, Dawson, Y. T., Canada.
Arthur Power Holloway, Steubenville, Ohio.
Clarence Decatur Howe, Waltham, Mass.
George William Howson, Jr., Berkeley, Cal.
Leigh Patterson Jerrard, Madison, Wis.
Ridgely Casey Lilly, Vicksburg, Miss.
Arthur Keddie Macfarlane, Benicia, Cal.
Alexander Mischke, Warsaw, Russia.
Christopher George Morrison, Albay, Philippine Islands.
Kenneth Howard Osborn, Rochester, N. Y.
Harold Whitney Roberts, New York City.
George Schobinger, Chicago, Ill.
Harold Anthony Sweetland, Providence, R. I.
Roger Wolcott Toll, Denver, Colo.
Charles Henry Ward, 2d, Middletown, R. I.
Edward Ashton Ward, Newark, N. J.
Frederick Sampson Weston, Middleboro, Mass.

The Secretary announced the transfer of the following candidates by the Board of Direction on October 5th, 1909:

From Associate Member to Member.

John Turney Fetherston, New Brighton, N. Y.
William Cushing Hammatt, San Francisco, Cal.
John Brown Lindsey, Jr., Gautier, Miss.
Thomas Herbert Means, Fallon, Nev.
Percy Hartshorne Wilson, Philadelphia, Pa.
Clarence George Wrentmore, Manila, Philippine Islands.

From Junior to Associate Member.

Max John Bartell, San Francisco, Cal.
George Noble Copley, Galveston, Tex.
Clinton Alonzo Curtis, Fort Edward, N. Y.

Charles Algernon Engle, Twin Falls, Idaho.
Thomas Fleming, Jr., Harrisburg, Pa.
Ralph Hamlin, St. Paul, Minn.
Henry Alexander Leeuw, Brooklyn, N. Y.
Birtram Willard Ransom, Youngstown, Ohio.
George Arthur Sisson, Vallejo, Cal.
Charles Harold Splitstone, East Orange, N. J.
Harry Percival Wilson, Worcester, Mass.

The Secretary announced the following deaths:

Henry Waller Brinckerhoff, elected Member, November 7th, 1883; died September 7th, 1909.

James Henry Covode, elected Member, April 2d, 1890; died September 9th, 1909.

Charles Morton, elected Member, October 2d, 1895; died August 28th, 1909.

Joseph Nelson Tubbs, elected Member, September 3d, 1884; died September 20th, 1909.

Carl William Birch-Nord, elected Junior, October 30th, 1906; Associate Member, November 4th, 1908; died September 15th, 1909.

Adjourned.

OF THE BOARD OF DIRECTION

(Abstract)

October 5th, 1909.—President Bates in the chair; Chas. Warren Hunt, Secretary; and present, also, Messrs. Andrews, Brackett, Endicott, Gardner, Harrison, Hodgdon, Horton, Noble, Pegram, Schneider, Stearns, Stuart, Swensson, Thompson, Tillson, and Wilkins.

The following Resolution was adopted by letter-ballot of the whole Board of Direction:

Resolved, that briefs of the Minutes of the meetings of the Board, to contain all business, except that which relates to routine, be forwarded to members of the Board of Direction for their information.

The following Resolution was adopted by letter-ballot of the whole Board of Direction:

Whereas, the attendance of Non-resident Members of the Board at its Meetings, as shown by statistics for the past 7 years, has not been as large as it should have been, especially from Districts Nos. 5, 6, and 7, and whereas it is extremely desirable that such attendance be increased if possible,

Resolved, that, beginning with the October, 1909, Board Meeting, and for a period of one year, the transportation of each Non-resident Member of the Board be paid at the rate of three cents per mile (on established mileage) for attendance at any of its meetings, except those held at the Annual Meeting and at the Annual Convention, and be it further

Resolved, that it is the sense of the Board that under these conditions it is the duty of each Non-resident Member of the Board to be present at at least three (3) of its Meetings during each year.

Ballots for Membership were canvassed, resulting in the election of 27 Members, 33 Associate Members, and 22 Juniors, the transfer of 6 Associate Members to the grade of Member, and 11 Juniors to the grade of Associate Member.

Applications were considered, and other routine business was transacted.

Adjourned.

ANNOUNCEMENTS

The House of the Society is open from 9 A. M. to 10 P. M., every day, except Sundays, Fourth of July, Thanksgiving Day, and Christmas Day.

MEETINGS

November 3d, 1909.—8.30 P. M.—At this meeting two papers will be presented for discussion as follows: "The Reinforced Concrete Wharf of the United Fruit Company at Bocas del Toro, Panama," by T. Howard Barnes, M. Am. Soc. C. E., and "River Protection Work on the Kansas City Southern Railway Near Braden, Okla.," by J. A. Lahmer, M. Am. Soc. C. E.

Mr. Barnes' paper was printed in *Proceedings* for September, 1909, and Mr. Lahmer's paper is printed in this number of *Proceedings*.

November 17th, 1909.—8.30 P. M.—At this meeting two papers will be presented for discussion as follows: "The Outlet Control of Little Bear Valley Reservoir," by F. E. Trask, M. Am. Soc. C. E., and "Water Supply for the Lock Canal at Panama," by Julio F. Sorzano, M. Am. Soc. C. E.

Mr. Trask's paper was printed in *Proceedings* for September, 1909, and Mr. Sorzano's paper is printed in this number of *Proceedings*.

December 1st, 1909.—8.30 P. M.—At this meeting a paper by James H. Brace and Francis Mason, Members, Am. Soc. C. E., entitled "The New York Tunnel Extension of the Pennsylvania Railroad: The Cross-Town Tunnels," will be presented for discussion.*

This paper is printed in this number of *Proceedings*.

December 15th, 1909.—8.30 P. M.—A paper by James H. Brace, Francis Mason, and S. H. Woodard, Members, Am. Soc. C. E., on "The New York Tunnel Extension of the Pennsylvania Railroad: The East River Tunnels," will be presented for discussion.*

This paper is printed in this number of *Proceedings*.

* These papers are a continuation of the papers by Gen. C. W. Raymond, M. Am. Soc. C. E., and Alfred Noble, Past-President, Am. Soc. C. E., published in the September number of *Proceedings*. At the meeting of December 15th there will also be presented a paper by Henry Japp, M. Am. Soc. C. E., describing the Contractor's Plant on the East River Tunnels. This paper will be published in the November number of *Proceedings*.

LIST OF NOMINEES FOR THE OFFICES TO BE FILLED AT THE ANNUAL ELECTION, JANUARY 19th, 1910.

The following list of nominees for the offices to be filled at the Annual Meeting, January 19th, 1910, received from the Nominating Committee, was presented to the Board of Direction at its meeting on August 31st, 1909. The list has already been mailed to all Corporate Members:

For President, to serve one year:

JOHN A. BENSEL, New York City.

For Vice-Presidents, to serve two years:

HUNTER MCDONALD, Nashville, Tenn.
JOHN T. FANNING, Minneapolis, Minn.

For Treasurer, to serve one year:

JOSEPH M. KNAP, New York City.

For Directors, to serve three years:

HORACE LOOMIS, New York City.....District No. 1
WILLIAM E. BELKNAP, New York City.....District No. 1
GEORGE A. KIMBALL, Boston, Mass.....District No. 2
PERCIVAL ROBERTS, JR., Philadelphia, Pa.....District No. 4
CHARLES F. LOWETH, Chicago, Ill.....District No. 5
VIRGIL G. BOGUE, San Francisco, Cal.....District No. 7

MEETINGS OF THE SAN FRANCISCO ASSOCIATION OF MEMBERS, AM. SOC. C. E.

The San Francisco Association of Members of the American Society of Civil Engineers holds regular bi-monthly meetings with banquet, and weekly informal luncheons. The former are held at 6 P. M., at the Fairmont Hotel, on the third Friday of February, April, June, August, October, and November, and also on the third Wednesday of December, the latter being the Annual Meeting of the Association.

Informal luncheons are held at 12.15 P. M. every Wednesday, and the place of meeting may be ascertained by communicating with the Secretary of the Association, E. T. Thurston, Jr., Assoc. M. Am. Soc. C. E., 623 Wells Fargo Building.

The by-laws of the Association provide for the extension of hospitality to any members of the Society who may be temporarily in San Francisco, and any such member will be gladly welcomed as a guest of the Association at any of the above meetings, if he will notify the Secretary that he is in San Francisco.

PAPERS AND DISCUSSIONS

The first three volumes of *Transactions* for 1909 (Volumes LXII, LXIII, and LXIV) have been issued. The fourth volume will be issued on December 31st.

It is hoped that members and others who take part in the discussion of the papers presented will revise their remarks promptly, and that all written communications from those who cannot attend the meetings will be sent in at the earliest possible date after the issue of a paper in *Proceedings*. The issue of volumes of *Transactions* is dependent on the closing of discussions, and the co-operation of the membership will now be more necessary in this matter than heretofore, because four volumes are issued during the year, instead of two, and, to accomplish this, a definite date of issue for each has to be established.

All papers accepted by the Publication Committee are classified by the Committee with respect to their availability for discussion at meetings.

Papers, which, from their general nature, appear to be of a character suitable for oral discussion, will be published as heretofore in *Proceedings*, and set down for presentation to a future meeting of the Society, and, on these, oral discussion, as well as written communications, will be solicited.

All papers which do not come under this heading, that is to say, those which, from their mathematical or technical nature, in the opinion of the Committee, are not adapted to oral discussion, will not be scheduled for presentation to any meeting. Such papers will be published in *Proceedings* in the same manner as those which are to be presented at meetings, but written discussions, only, will be requested for subsequent publication in *Proceedings* and with the paper in the volumes of *Transactions*.

PRIVILEGES OF ENGINEERING SOCIETIES EXTENDED TO MEMBERS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

Members of the American Society of Civil Engineers will be welcomed by the following Engineering Societies, both to the use of their Reading Rooms and at all Meetings:

American Institute of Mining Engineers, 29 West Thirty-ninth Street, New York City.

Associação dos Engenheiros Civis Portuguezes, Lisbon, Portugal.

Australasian Institute of Mining Engineers, Melbourne, Victoria, Australia.

Boston Society of Civil Engineers, 715 Tremont Temple, Boston, Mass.

Brooklyn Engineers' Club, 197 Montague Street, Brooklyn, N. Y.

Canadian Society of Civil Engineers, 413 Dorchester Street, West, Montreal, Que., Canada.

Civil Engineers' Society of St. Paul, St. Paul, Minn.

Cleveland Engineering Society, 718 Caxton Building, Cleveland, Ohio.

Cleveland Institute of Engineers, Middlesbrough, England.

Colorado Association of Members, Am. Soc. C. E., H. J. Burt, Secy., 235 Equitable Building, Denver, Colo.

Engineers' and Architects' Club of Louisville, Ky., 303 Norton Building, Fourth and Jefferson Streets, Louisville, Ky.

Engineers' Club of Baltimore, Baltimore, Md.

Engineers' Club of Central Pennsylvania, Corner Second and Walnut Streets, Harrisburg, Pa.

Engineers' Club of Minneapolis, 17 South Sixth Street, Minneapolis, Minn.

Engineers' Club of Philadelphia, 1317 Spruce Street, Philadelphia, Pa.

Engineers' Club of St. Louis, 3817 Olive Street, St. Louis, Mo.

Engineers' Club of Toronto, 96 King Street, West, Toronto, Ont., Canada.

Engineers' Society of Western Pennsylvania, 803 Fulton Building, Pittsburg, Pa.

Institute of Marine Engineers, 58 Romford Road, Stratford, London, E., England.

Institution of Engineers of the River Plate, Buenos Aires, Argentine Republic.

Institution of Naval Architects, 5 Adelphi Terrace, London, W. C., England.

Junior Institution of Engineers, 39 Victoria Street, Westminster, S. W., London, England.

Koninklijk Instituut van Ingenieurs, The Hague, The Netherlands.

Louisiana Engineering Society, 604 Tulane-Newcomb Building, New Orleans, La.

Memphis Engineering Society, Memphis, Tenn.

Midland Institute of Mining, Civil and Mechanical Engineers, Sheffield, England.

Montana Society of Engineers, Butte, Montana.

North of England Institute of Mining and Mechanical Engineers, Newcastle-upon-Tyne, England.

Oesterreichischer Ingenieur- und Architekten-Verein, Eschenbachgasse 9, Vienna, Austria.

Pacific Northwest Society of Engineers, 617-618 Pioneer Building, Seattle, Wash.

Rochester Engineering Society, Rochester, N. Y.

Sachsischer Ingenieur- und Architekten-Verein, Dresden, Germany.

Sociedad Colombiana de Ingenieros, Bogota, Colombia.

Societe des Ingenieurs Civils de France, 19 Rue Blanche, Paris, France.

Society of Engineers, 17 Victoria Street, Westminster, S. W., London, England.

Svenska Teknologforeningen, Brunkebergstorg 18, Stockholm, Sweden.

Tekniske Forening, Vestre Boulevard 18–1, Copenhagen, Denmark.

Western Society of Engineers, 1737 Monadnock Block, Chicago, Ill.

SEARCHES IN THE LIBRARY

In January, 1902, the Secretary was authorized to make searches in the Library, upon request, and to charge therefor the actual cost to the Society for the extra work required. Since that time many searches have been made, and bibliographies and other information on special subjects furnished.

The resulting satisfaction, to the members, who have made use of the resources of the Society in this manner, has been expressed frequently, and leaves little doubt that, if it were generally known to the membership that such work would be undertaken, many would avail themselves of it.

The cost is trifling compared with the value of the time of an engineer who looks up such matters himself, and the work can be performed quite as well, and much more quickly, by persons familiar with the Library.

In asking that such work be undertaken, members should specify clearly the subject to be covered, and whether references to general books only are desired, or whether a complete bibliography, involving search through periodical literature, is desired.

In reference to this work, the Appendix* to the Annual Report of the Board of Direction for the year ending December 31st, 1906, contains a summary of all searches made to that date.

* *Proceedings*, Vol. XXXIII, p. 20 (January, 1907).

ACCESSIONS TO THE LIBRARY

(From September 8th to October 11th, 1909)

DONATIONS*

THE ELEMENTS OF MECHANICS OF MATERIALS.

A Text Book for Students in Engineering Courses. By C. E. Houghton. Cloth, 8½ x 6 in., illus., 7 + 186 pp. New York, D. Van Nostrand Company, 1909. $2.00.

The author, as stated in the preface, has designed this book to be an elementary textbook for students in the engineering courses in colleges and universities, and not a treatise on the mechanics of materials. Extreme mathematical treatment of the subject is said to have been avoided, the higher mathematics being used only for clearness of demonstration. Review questions are given at the end of each chapter, covering the more important parts of the subjects discussed and the problems illustrating such subjects, the solution of one problem of each type being given in order to show the application of the general formulas. Tables giving values of engineering constants of materials and formulas are contained in the appendix. It is stated that the notation has been made uniform with that of Professor Merriman's works, in order that his treatise on the subject may be used as a reference book. The Contents are: Applied Mechanics; Applications; Beams; Torsion; The Elastic Curve; Long Columns; Combined Stresses; Compound Bars and Beams; Index.

AMERICAN STREET RAILWAY INVESTMENTS, 1909.

Published Annually by the Publishers of the *Electric Railway Journal* for the Use of Bankers, Brokers, Capitalists, Investors, and Street and Interurban Railway Companies. Sixteenth Annual Volume. Cloth 13 x 10 in., illus., 40 + 498 pp. New York, McGraw Publishing Company, 1909. $5.00.

This publication contains the names of the street railway companies in the United States, Canada, West Indies, Hawaii, and the Philippine Islands, arranged alphabetically under State and city. A short history of each road is given, together with the amount of its capital stock, funded debt, and plant. The names and addresses of the officers of each company and also the addresses of the general offices, power stations, and repair shops, are given. At the end of each report is the date on which such information was obtained. Maps of the various railway systems and of cities showing the railway lines are included, as are lists of the names and addresses of the officers and members of the National and State railroad commissions, together with those of the officers of various street and interurban railway associations.

THE COMMON METALS (NON-FERROUS).

The Metallurgy of Copper, Tin, Zinc, Lead, Antimony, Aluminium, and Nickel. By A. Humboldt Sexton and John S. G. Primrose. Cloth, 9 x 6 in., illus., 8 + 483 + 9 pp. Manchester, England, The Scientific Publishing Company. 7 shillings 6 pence net.

It is stated in the preface that in preparing this textbook, in which accuracy, suitable selection of material, and conciseness are essential, the authors have drawn on all authorities within their reach, as well as on their own experience. The book, as stated, is intended only for students who, if they wish to carry their studies further or are engaged in practical metallurgical work, must refer to the larger books mentioned in the bibliography in Appendix IV. The Contents are: Properties of Copper; Copper Minerals; The Principles of Copper Smelting; The Welsh Copper Process, etc.; Ore Dressing; Modern Methods of Copper Smelting; Matte Smelting in Blast Furnaces; Running for Coarse Copper in Reverberatory Furnaces; Copper Refining; Wet or Leaching Processes; Electro-Deposition of Copper; Properties of Tin; Working Alluvial Deposits; Tin Smelting; Refining Tin; Properties of Zinc; Dressing Zinc Ores; The English, Belgian, Silesian, and Rhenish Processes; Effect of Impurities; Properties of Lead; Preparation of Lead Ores; Smelting in Hearths;

*Unless otherwise specified, all books in this list have been donated by the publisher.

Roasting Lead Ores; Blast Furnace Smelting; Blast Furnaces for Lead Smelting; Products of Lead Smelting Fume Recovery; Wet and Electrolytic Processes; Softening and Refining Lead; Concentration of Silver in Lead; Cupellation; Electrolytic Refining of Lead; Antimony; Properties of Aluminium; Process for the Preparation of Aluminium; Properties of Nickel; Dry Methods for the Extraction of Nickel, etc.; Appendix I, Physical Constants of Metals; Appendix II, Thermo Chemical Data; Appendix III, Factors in Calculating Blast Furnace Charges; Appendix IV, Bibliography; Index.

DE SCHEIDING VAN MAAS EN WAAL.

Onder Verlegging van de Uitmonding der Maas Naar den Amer. Bewerkt door den Ingenieur van den Waterstaat, M. C. E. Bongaerts. Cloth, 12 x 9½ in., illus., 11 + 356 pp. Haarlem, Uitgegeven door het Ministerie van Waterstaat, 1909. (Presented by R. P. J. Tutein Nolthenius, M. Am. Soc. C. E.)

This book, which is published by order of the Dutch Government, contains a description of the separation of the Rivers Maas and Waal, in The Netherlands, at a cost of about $10 000 000. The principal part of this engineering work consisted of the formation of a new river. The Contents are: Geschiedkundige Inleiding; Plannen tot Verlegging van den Maasmond; Beschrijving van de Uitgevoerde Werken; Administratieve Regelingen; Recapitulatie van de Kosten; Personeel; Bijlagen; Platen; Tekstfiguren.

HENDRICKS' COMMERCIAL REGISTER OF THE UNITED STATES.

For Buyers and Sellers. Eighteenth Annual Edition. Cloth, 10½ x 7½ in., illus., 87 + 1220 pp. New York, Samuel E. Hendricks Co., 1909. $10.00.

It is stated that this publication is a complete index, containing names and addresses and business classifications, of the architectural, mechanical, engineering, contracting, electrical, railroad, iron, steel, hardware, mining, mill, quarrying, exporting, and kindred industries. Full lists of manufacturers of and dealers in everything used in the manufacture of material, machinery, and apparatus for these vast industries are also given. The contents are classified by subject, under which are given, in alphabetical order, and, in some cases, by States, the names and addresses of firms dealing in that particular article. There is an index of eighty-seven pages.

A TREATISE ON MASONRY CONSTRUCTION.

By Ira Osborn Baker, M. Am. Soc. C. E. Tenth Edition, Re-Written and Enlarged. Cloth, 9 x 6 in., illus., 15 + 745 pp. New York, John Wiley & Sons; London, Chapman & Hall, Limited, 1909. $5.00.

Two previous editions of this book have been published, one in 1889 and the other in 1899. It is stated that the extensive use of plain concrete and the introduction of reinforced concrete have made this, the third edition, necessary; therefore the book has been entirely re-written in order to bring the subject-matter up to date. Numerous changes and additions have been made in the text, but most of the new matter will be found in the chapter on Plain Concrete, in the three new chapters on Reinforced Concrete, Concrete Building-Blocks, and Elastic Arch, and in connection with the new structures described. The number of the latter has been increased in this edition, and it is believed that they represent the latest practice. In the preface to the first edition, the author states that the object of this work has been to develop principles and methods of construction and to give such examples as will illustrate them, his constant aim having been to present the subject clearly and concisely. The Contents are: I. The Materials: Natural Stone; Building Brick; Common and Hydraulic Lime; Hydraulic Cement; Sand, Gravel, and Broken Stone. II. Preparing and Using the Materials: Lime and Cement Mortar; Plain Concrete; Reinforced Concrete; Concrete Building-Blocks and Artificial Stone; Stone Cutting; Stone Masonry; Brick Masonry. III. Foundations: Introductory; Ordinary Foundations; Pile Foundations; Foundations Under Water. IV. Masonry Structures: Masonry Dams; Retaining Wall; Bridge Abutments; Bridge Piers; Culverts; Voussoir Arch; Elastic Arch. Specifications: For Cement; For Concrete; For Masonry; Index.

STRUCTURAL DETAILS

Or, Elements of Design in Heavy Framing. By Henry S. Jacoby, Assoc. Am. Soc. C. E. Cloth, 9 x 6 in., illus., 9 + 368 pp. New York, John Wiley & Sons; London, Chapman & Hall, Limited, 1909. $2.25 net.

The title of this volume is declared in the preface to correspond to a course of instruction conducted by the author in the College of Civil Engineering of Cornell University, in which the students receive their first instruction in the application of the principles of mechanics to structures. Chapter I is said to contain the digested results of extensive experimental investigation and research, and numerous references to engineering periodicals and transactions are made a prominent feature throughout the volume. An attempt has been made, the preface states, to present the subjects of joints and fastenings in a systematic manner, and to include practical information on timber construction. The Contents are: Fastenings Used in Framing; Joints Used in Framing; Wooden Beams and Columns; Wooden Roof Trusses; Examples of Framing in Practice; Timber Tests and Unit-Stresses; Index.

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The Lead and Zinc Pigments. By Clifford Dyer Holley. John Wiley & Sons, New York; Chapman & Hall, Ltd., London, 1909.

Gear-cutting Machinery, Comprising a Complete Review of Contemporary American and European Practice, Together with a Logical Classification and Explanation of the Principles Involved. By Ralph E. Flanders. John Wiley & Sons, New York; Chapman & Hall, Ltd., London, 1909.

The Gas, Petrol and Oil Engine. Vol. 1, Thermo-dynamics of the Gas, Petrol and Oil Engine Together with Historical Sketch. By Dugald Clerk. New and Revised Edition. John Wiley & Sons, New York, 1909.

The Dressing of Minerals. By Henry Louis. Longmans, Green & Co., New York; Edward Arnold, London, 1909.

Die Förderung von Massengütern. By Georg von Hauffstengel. Band I, Bau und Berechnung der Stetig Arbeitenden Förderer; Band II, Förderer für Einzellasten. 2 vol. Julius Springer, Berlin, 1908–09.

Robert Fulton and the *Clermont*. By Alice Crary Sutcliffe. Century Co., New York, 1909.

Timbering and Mining: A Treatise on Practical American Methods. By William H. Storms. McGraw-Hill Book Co., New York, 1909.

Ore Dressing. By Robert H. Richards. Vols. III and IV. McGraw-Hill Book Co., New York, 1909.

Lloyd's Register of British and Foreign Shipping from 1st of July, 1909, to 30th of June, 1910. Vol. 1, Steamers, Sailing Vessels and Owners; Vol. 2, Appendix. Committee of Lloyd's Register and Foreign Shipping, London, 1909.

The Safety of British Railways: or, Railway Accidents, How Caused and How Prevented. By H. Raynar Wilson. P. S. King & Son, London, 1909.

The Adventures of a Civil Engineer, Fifty Years on Five Continents. By C. O. Burge. Alston Rivers, Ltd., London, 1909.

Strength of Materials. By Arthur Morley. Longmans, Green & Co., London, New York, and Calcutta, 1908.

History of New York Shipyards. By John H. Morrison. W. F. Sametz & Co., New York, 1909.

Remaking the Mississippi. By John Lathrop Mathews. Houghton, Mifflin Co., Boston; The Riverside Press, Cambridge, 1909.

The Manufacture of Explosives, a Theoretical and Practical Treatise on the History, the Physical and Chemical Properties and the Manufacture of Explosives. By Oscar Guttmann. 2 vol. Whittaker & Co., London, 1895.

Report of the Municipal Art Commission for the City of Los Angeles, California, Made to the Mayor, the City Council, the Board of Public Works. By F. W. Blanchard, President, John W. Mitchell, Secretary, John Parkinson, Mrs. W. J. Washburn, Mrs. Sumner P. Hunt, Municipal Art Commission. William J. Porter, Los Angeles, 1909.

The Report of Charles Mulford Robinson Regarding the Civic Affairs of Santa Barbara, California, also the Report of the Committee of Eleven on the Improvement of the City Streets. Printed for the Civic League by *The Independent*, 1909.

Sewage Disposal Works, a Guide to the Construction of Works for the Prevention of the Pollution by Sewage of Rivers and Estuaries. By W. Santo Crimp. Charles Griffin & Co., London, 1890.

The Universal Directory of Railway Officials, 1909. Compiled from Official Sources under the Direction of S. Richardson Blundstone. The Directory Publishing Co., Limited, London; A. Fenton Walker, 143 Liberty Street, New York City, 1909.

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DEATHS

BIRCH-NORD, CARL WILLIAM. Elected Junior, October 30th, 1906; Associate Member, November 4th, 1908; died September 15th, 1909.

BRINCKERHOFF, HENRY WALLER. Elected Member, November 7th, 1883; died September 7th, 1909.

COVODE, JAMES HENRY. Elected Member, April 2d, 1890; died September 9th, 1909.

PARKER, WILLIAM. Elected Member, March 7th, 1900; died September 30th, 1909.

TUBBS, JOSEPH NELSON. Elected Member, September 3d, 1884; died September 20th, 1909.

Total Membership of the Society, October 12th, 1909, 5 166.

MONTHLY LIST OF RECENT ENGINEERING ARTICLES OF INTEREST

(September 8th to October 11th, 1909)

NOTE.—*This list is published for the purpose of placing before the members of the Society, the titles of current engineering articles, which can be referred to in any available engineering library, or can be procured by addressing the publication directly, the address and price being given wherever possible.*

LIST OF PUBLICATIONS

In the subjoined list of articles, references are given by the number prefixed to each journal in this list:

(1) *Journal*, Assoc. Eng. Soc., 31 Milk St., Boston, Mass., 30c.
(2) Proceedings, Engrs. Club of Phila., 1317 Spruce St., Philadelphia, Pa.
(3) *Journal*, Franklin Inst., Philadelphia, Pa., 50c.
(4) *Journal*, Western Soc. of Engrs., Monadnock Blk., Chicago, Ill.
(5) *Transactions*, Can. Soc. C. E., Montreal, Que., Canada.
(6) *School of Mines Quarterly*, Columbia Univ., New York City, 50c.
(7) *Technology Quarterly*, Mass. Inst. Tech., Boston, Mass., 75c.
(8) *Stevens Institute Indicator*, Stevens Inst., Hoboken, N. J., 50c.
(9) *Engineering Magazine*, New York City, 25c.
(10) *Cassier's Magazine*, New York City, 25c.
(11) *Engineering* (London), W. H. Wiley, New York City, 25c.
(12) *The Engineer* (London), International News Co., New York City, 35c.
(13) *Engineering News*, New York City, 15c.
(14) *The Engineering Record*, New York City, 12c.
(15) *Railroad Age Gazette*, New York City, 15c.
(16) *Engineering and Mining Journal*, New York City, 15c.
(17) *Electric Railway Journal*, New York City, 10c.
(18) *Railway and Engineering Review*, Chicago, Ill., 10c.
(19) *Scientific American Supplement*, New York City, 10c.
(20) *Iron Age*, New York City, 10c.
(21) *Railway Engineer*, London, England, 25c.
(22) *Iron and Coal Trades Review*, London, England, 25c.
(23) *Bulletin*, American Iron and Steel Assoc., Philadelphia, Pa.
(24) *American Gas Light Journal*, New York City, 10c.
(25) *American Engineer*, New York City, 20c.
(26) *Electrical Review*, London, England.
(27) *Electrical World*, New York City, 10c.
(28) *Journal*, New England Water-Works Assoc., Boston, Mass., $1.
(29) *Journal*, Royal Society of Arts, London, England, 15c.
(30) *Annales des Travaux Publics de Belgique*, Brussels, Belgium.
(31) *Annales* de l'Assoc. des Ing. Sortis des Ecoles Spéciales de Gand, Brussels, Belgium.
(32) *Mémoires et Compte Rendu des Travaux*, Soc. Ing. Civ. de France, Paris, France.
(33) *Le Génie Civil*, Paris, France.
(34) *Portefeuille Economiques des Machines*, Paris, France.
(35) *Nouvelles Annales de la Construction*, Paris, France.
(37) *Revue de Mécanique*, Paris, France.
(38) *Revue Générale des Chemins de Fer et des Tramways*, Paris, France.
(41) *Modern Machinery*, Chicago, Ill., 10c.
(42) *Proceedings*, Am. Inst. Elec. Engrs., New York City, 50c.
(43) *Annales des Ponts et Chaussées*, Paris, France.
(44) *Journal*, Military Service Institution, Governors Island, New York Harbor, 50c.
(45) *Mines and Minerals*, Scranton, Pa., 20c.
(46) *Scientific American*, New York City, 8c.
(47) *Mechanical Engineer*, Manchester, England.
(48) *Zeitschrift*, Verein Deutscher Ingenieure, Berlin, Germany.
(49) *Zeitschrift für Bauwesen*, Berlin, Germany.
(50) *Stahl und Eisen*, Düsseldorf, Germany.
(51) *Deutsche Bauzeitung*, Berlin, Germany.
(52) *Rigasche Industrie-Zeitung*, Riga, Russia.
(53) *Zeitschrift*, Oesterreichischer Ingenieur und Architekten Verein, Vienna, Austria.
(54) *Transactions*, Am. Soc. C. E., New York City, $5.
(55) *Transactions*, Am. Soc. M. E., New York City, $10.
(56) *Transactions*, Am. Inst. Min. Engrs., New York City, $5.

(57) *Colliery Guardian*, London, England.
(58) *Proceedings*, Eng. Soc. W. Pa., 803 Fulton Bldg., Pittsburg, Pa., 50c.
(59) *Transactions*, Mining Inst. of Scotland, London and Newcastle-upon-Tyne, England.
(60) *Municipal Engineering*, Indianapolis, Ind., 25c.
(61) *Proceedings*, Western Railway Club, 225 Dearborn St., Chicago, Ill., 25c.
(62) *Industrial World*, 59 Ninth St., Pittsburg, Pa.
(63) *Minutes of Proceedings*, Inst. C. E., London, England.
(64) *Power*, New York City, 20c.
(65) *Official Proceedings*, New York Railroad Club, Brooklyn, N. Y., 15c.
(66) *Journal of Gas Lighting*, London, England, 15c.
(67) *Cement and Engineering News*, Chicago, Ill., 25c.
(68) *Mining Journal*, London, England.
(70) *Engineering Review*, New York City, 10c.
(71) *Journal*, Iron and Steel Inst., London, England.
(73) *Electrician*, London, England, 18c.
(74) *Transactions*, Inst. of Min. and Metal., London, England.
(75) *Proceedings*, Inst. of Mech. Engrs., London, England.
(76) *Brick*, Chicago, Ill., 10c.
(77) *Journal*, Inst. Elec. Engrs., London, England.
(78) *Beton und Eisen*, Vienna, Austria.
(79) *Forscherarbeiten*, Vienna, Austria.
(80) *Tonindustrie Zeitung*, Berlin, Germany.
(81) *Zeitschrift für Architektur und Ingenieurwesen*, Wiesbaden, Germany.
(82) *Dinglers Polytechnisches Journal*, Berlin, Germany.
(83) *Progressive Age*, New York City, 15c.
(84) *Le Ciment*, Paris, France.
(85) *Proceedings*, Am. Ry. Eng. and M. of W. Assoc., Chicago, Ill.
(86) *Engineering-Contracting*, Chicago, Ill., 10c.
(87) *Roadmaster and Foreman*, Chicago, Ill., 10c.
(88) *Bulletin* of the International Ry. Congress Assoc., Brussels, Belgium.
(89) *Proceedings*, Am. Soc. for Testing Materials, Philadelphia, Pa.
(90) *Transactions*, Inst. of Naval Archts., London, England.
(91) *Transactions*, Soc. Naval Archts. and Marine Engrs., New York City.
(92) *Bulletin*, Soc. d'Encouragement pour l'Industrie Nationale, Paris, France.
(93) *Revue de Métallurgie*, Paris, France, 4 fr. 50.
(94) *The Boiler Maker*, New York City, 10c.
(95) *International Marine Engineering*, New York City, 20c.
(96) *Canadian Engineer*, Toronto, Canada, 15c.
(97) *Turbine*, Berlin, Germany, 1 Mark.

LIST OF ARTICLES

Bridge.

Reinforced Concrete Trestles.* (Report of Comm. Amer. Ry. Eng. and M. of W. Assoc.) (85) Vol. 10, Pt. 2.
Impact Tests.* (Report of Comm. Amer. Ry. Eng. and M. of W. Assoc.) (85) Vol. 10, Pt. 2.
Wooden Bridges and Trestles.* (Report of Comm. Amer. Ry. Eng. and M. of W. Assoc.) (85) Vol. 10, Pt. 1.
Foundation Work for C. and N. W. Railroad Bridge across the Mississippi River at Clinton, Iowa.* M. Deutsch. (6) July.
Strengthening the Track and the Bridges with a View to Increasing the Speed of Trains (Subject for Discussion at the Eighth Session of the Railway Congress).* (88) Sept.
The Makatote Viaduct.* (11) Sept. 3.
The Widening of Blackfriars Bridge.* (11) Sept. 10.
Methods and Cost of Constructing a Reinforced Concrete Arch, using Chutes to Deposit the Concrete. (86) Sept. 15.
The 180-Ft. Stone Arch Bridge at Wiesen, Switzerland.* (13) Sept. 16.
Great Engineering Works on the Canadian Pacific Railway (Lethbridge Viaduct).* J. E. Schwitzer. (Paper read before the Brit. Assoc.) (96) Sept. 17; (13) Sept. 23; (11) Sept. 24.
Some Concrete Culverts and Small Bridges on the Slateford-Hopatcong Cut-Off of the Delaware, Lackawanna & Western R. R.* (13) Sept. 30.
The Design and Construction of the Manhattan Bridge.* (14) Oct. 2.
Erection of the Fades Viaduct.* (46) Oct. 9.
Reconstruction of Approach Spans, Poughkeepsie Bridge.* (14) Oct. 9.
The Calculation of Stresses in Stiffening Trusses.* F. E. Turneaure. (From Appendix to Report on Manhattan Bridge). (14) Oct. 9.
Le Pont Tournant Kaiser Wilhelm à Wilhelmshafen (Allemagne).* (33) Sept. 11.

*Illustrated.

Bridge—(Continued).

Die Duisburg-Ruhrorter Klappbrücken nebst den Zugehörigen Festen Strassenbrucken.* Ottmann and Loebell. (49) Vol. 8 and 9, 1909.
Strassenbrücke über den Schussenkanal in Ravensburg.* H. Pilgrim. (78) Serial beginning Sept. 1.
Zur Materialausnützung des Eisernen Dachbinders. Fritz Weiss. (53) Sept. 10.
Nickelstahl für Eisenbrücken. (50) Sept. 15.

Electrical.

Wireless Telephony.* C. Tissot. (From *La Technique Moderne.*) (88) Sept.
Atmospheric Loss off Wires under Direct Current Pressures.* E. A. Watson. (Paper read before the Brit. Assoc.) (73) Sept. 3.
Relations between Output, Losses, Dimensions, Weight and Cost of Transformers.* E. G. Reed. (27) Sept. 9.
A South American Gas-Engine-Driven Central Station.* J. P. Stone. (27) Sept. 9.
A New Electrical Generating Station at Buenos Ayres.* (12) Sept. 10.
Electrical Equipment of the Great Western Power Company.* (27) Sept. 16.
The Starting Torque of Three-Phase Motors with Squirrel-Cage Rotors.* Max Kloss. (From the *Electron.*) (73) Sept. 17.
Commutation in Continuous-Current Machines.* H. E. Mellor. (47) Serial beginning Sept. 17.
Central Electric Plants in Small Towns.* James S. Knowlson. (Abstract of a paper read before the National Elec. Light Assoc.) (13) Sept. 23.
Oakland (Cal.) Steam Auxiliary Station for Great Western Power Company.* (27) Sept. 23.
Telegraph and Telephone Systems as Affected by Alternating-Current Lines.* J. B. Taylor. (42) Oct.
The Electricity Supply Stations of the New York Edison Company.* (73) Serial beginning Oct. 1.
Ammeter for the Accurate Measurement of Large Alternating Currents.* E. F. Northrup. (Abstract of paper read before the Amer. Electrochemical Soc.) (73) Oct. 1.
The Phenomena of Resonance in Electric Lines.* J. Dalemont. (96) Oct. 1.
The Brusis Central Station.* J. B. Van Brussel. (64) Oct. 5.
Report of the Committee on Power Generation.* (Abstract of report read before the Amer. Street and Interurban Ry. Eng. Assoc.) (17) Oct. 7.
Electric Light and Exhaust Steam-Heating Station at West Chester, Pa.* (27) Oct. 7.
Gas Engine Driven Electric Plants versus Electric Motors Fed from Central Stations. R. D. Donaldson. (27) Oct. 7.
Elektrisch Betriebene Kohlenkipperanlage am Rothesay-Dock bei Glasgow.* W. Schlachter. (48) Serial beginning Aug. 7.
Wesen und Wirtschaftlichkeit Neuerer Elektrischer Starklichtquellen.* P. Heyck. (48) Aug. 7.
Uber Hochspannungsfreileitungen.* Ludwig Kallir. (53) Serial beginning Aug. 27.
Elektrizität und Dampf. (80) Sept. 21.

Marine.

Hydroplanes or Skimmers.* *Sir* John Thornycroft. (Paper read before the Brit. Assoc.) (11) Sept. 3.
Experimental Spur-Wheel Reduction Gear for High-Speed Steam-Turbines.* (11) Sept. 17.
The Number of Shafts for Turbine Vessels. (11) Sept. 24.
The Stability of Floating Docks.* Bernard C. Laws, M. I. N. A. (10) Oct.
The Design of Turning Engines.* Edward M. Bragg. (95) Serial beginning Oct.
Dutch Marine Suction-Gas Plants.* F. Muller Van Brakel. (95) Serial beginning Oct.
Train-Ferry Steamer *Fabius* for Northern Nigeria.* (11) Oct. 1.
The Uses of Electricity on Board Ship. John McLaren. (Abstract of paper read before the Inst. of Mar. Engrs.) (47) Oct. 1.
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The Mexican Coke Industry.* R. D. Martin. (45) Oct.
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Die Beziehungen der Gross-Dampfturbine zur Elektrotechnik. v. Groddeck. (Abstract from the *Elektrotechnische Zeitschrift.*) (97) Aug. 5.
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The Estimation of Sulpho and Ferrocyanides, etc., in Cyanide Solutions Containing Copper. Leonard M. Green. (74) Vol. 18.
Notes on Tin Dressing.* H. W. Hutchin. (74) Vol. 18.
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The Bessemerising of Hardhead.* Donald M. Levy and D. Ewen. (74) Vol. 18.

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A Study of Rail Pressures and Stresses in Track Produced by Different Types of Steam Locomotives when Rounding Various Degree Curves at Different Speeds.* E. E. Stetson. (85) Vol. 10, Pt. 2.

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Note sur les Chemins de Fer de l'Afrique Occidentale Française.* R. Godfernaux. (38) Sept.

Amélioration de la Traction à Air Comprimé par l'Utilisation de la Chaleur de Compression de l'Air.* L. Pierre-Guédon. (33) Sept. 11.

La Traction Electrique à Courant Continu et à Intensité Constante.* (33) Sept. 18.

Neue Elektrische Güterzuglokomotive der Italienischen Staatsbahnen.* K. von Kando. (48) Serial beginning Aug. 7.

Die Rillenschiene, Ihre Entstehung und Entwicklung.* Philipp Fischer. (50) Serial beginning Aug. 11.

Wagen- und Lokomotivschuppen der Salzburger Eisenbahn- und Tramway-Gesellschaft.* Franz Böhm. (78) Sept. 1.

Die Verladung von Massengütern im Eisenbahnbetrieb.* Prof. Aumund. (48) Serial beginning Sept. 4.

Versuche mit Durchgehenden Selbsttätigen Bremser bei Güterzügen.* Johann Rihosek. (53) Serial beginning Sept. 24.

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*Illustrated.

Railroad, Street.

The Central Repair Depot of the London County Council Tramways. W. E. Ireland. (Abstract of paper read before the Mun. Tramways Assoc.) (73) Sept. 24.
The Denver City Tramway System.* (17) Oct. 2.
The Stations of the Washington St. Tunnel, Boston, Mass.* Laurence B. Manley. (13) Oct. 7.
Neuere Systemformen im Eisenhochbau.* Franz Czech. (50) Aug. 25.

Sanitary.

Sewage Disposal Plant at Seneca Falls, New York.* (14) Sept. 11.
Some Methods of Sewer Tunnelling in Cities.* Alfred E. Snape, Assoc. M. Inst. C. E. (Abstract of paper read before the Inst. of County and Mun. Engrs.) (86) Sept. 15.
Data on Methods and Cost of Cleaning Catch Basins, Obtained from Practice in the Cities of Boston and Worcester, Mass.* (Abstract from Report to the Boston Finance Comm.) (86) Sept. 15.
The New Sprinkling Filter at Reading.* (14) Sept. 25.
Early History of Sewers of Memphis, Tenn. A. T. Bell. (From *Civic Affairs.*) (60) Oct.
The Cost of Concrete *vs.* Brick Sewers.* (86) Oct. 6.
A Committee Report on Sewage Disposal in the United States. (86) Oct. 6.

Structural.

Wood Preservation.* (Report of Comm. Amer. Ry. Eng. and M. of W. Assoc.) (85) Vol. 10, Pt. 1.
Buildings.* (Report of Comm. Amer. Ry. Eng. and M. of W. Assoc.) (85) Vol. 10, Pt. 2.
Reinforced Concrete Foundations for Stamp Batteries.* S. J. Truscott and John P. Fuller. (74) Vol. 18.
The Setting of Portland Cement, with Description of Methods Adopted for Regulating the Same. H. K. G. Bamber. (Abstract of paper read at the Concrete Inst.) (21) Sept.
The Behaviour of Ductile Material under Torsional Strain. C. E. Larard, Assoc. M. Inst. C. E. (Paper read before the Brit. Assoc.) (11) Serial beginning Sept. 3.
The Wemlinger Steel Sheet Piling.* (20) Sept. 9.
Reinforced Concrete Columns.* W. Noble Twelvetrees, M. I. Mech. E. (11) Serial beginning Sept. 10.
The Design and Construction of Forms for Concrete Work.* (86) Serial beginning Sept. 22.
A Note on the Deflection due to Shear.* W. C. Popplewell. (12) Sept. 24.
Extended Plant of the American La France Fire Engine Company.* (14) Sept. 25.
Moments in Continuous Reinforced-Concrete Beams under Uniform Loading.* R. E. Spaulding. (13) Sept. 30.
The Theory of Impact and its Application to Testing Materials.* Harry D. Tiemann. (3) Serial beginning Oct.
Electricity in Building Operations.* (73) Oct. 1.
Progress in Methods of Testing Hydraulic Cements. (Abstract of report to the Inter. Assoc. for Testing Materials.) (14) Oct. 2.
The Emigrant Industrial Savings Bank Building, New York.* (14) Oct. 2.
Making Concrete Waterproof. Ira O. Baker, M. Am. Soc. C. E. (From *The Technograph.*) (13) Oct. 7; (14) Oct. 9.
Calibration of the 2 400 000-lb. Hydraulic Testing Machine of the Phœnix Iron Co., at Phœnixville, Pa., by Comparison with the Watertown Arsenal Emery Machine. (13) Oct. 7.
Structural Frames of the Open-Hearth Buildings at Gary.* (14) Oct. 9.
The Design of Members Subjected to Combined Stresses.* E. L. Hancock. (14) Oct. 9.
Reinforced Concrete Joists with Hollow Tile Fillers.* J. Norman Jensen. (14) Oct. 9.
Expériences sur la Décomposition des Mortiers par les Eaux Sulfatées.* J. Bied. (93) June.
Récherches sur la Conductibilité, la Porosité et la Perméabilité des Matières Réfractaires.* S. Wologdine. (93) June.
Peintures Sous-Marins et Peintures Ignifuges. Ch. Coffignier. (93) June.
La Section des Métaux du Laboratoire d'Essais du Conservatoire National des Arts et Métiers.* Pierre Breuil. (32) July.
Note sur les Injections de Coulis de Ciment dans les Maçonneries.* Suquet. (43) July.
Equilibre Elastique des Plaques Circulaires Epaisses. Mesnager. (43) July.

*Illustrated.

Structural—(Continued).

Essai de Dureté des Chaux et Ciments par la Méthode de la Bille son Emploi pour la Détermination de la Prise et du Durcissement.* Laborbe. (93) Sept.
Rahmenkonstruktionen und deren Berechnung im Neubau Konditerei Weitz in Düsseldorf.* E. Elwitz. (78) July 22.
Heben und Verschieben von Gebäuden.* F. M. Feldhaus. (53) July 30.
Die Eisenbeton-Konstruktionen der Städtischen Müllverbrennungs-Anstalt in Frankfurt a. M.* Kölle and others. (Extract of paper read before the Deutsche Beton-Verein.) (51) Serial beginning July 31.
Kalkaluminate und Mörtel. (80) Aug. 7.
Bericht des Meerwasserausschusses des Ziegelei-Berufsgenossenschaftes. (80) Aug. 14.
Schmelzversuche zum Nachweis des Einflusses der Chemischen Zusammensetzung auf Hydraulizität der Schlacken. H. Passow. (50) Aug. 18.
Ueber die Knickfestigkeit von Stäben Veränderlichen Trägheitsmomentes. Fr. Engesser. (53) Aug. 20.
Bericht des Bindezeitausschusses des Vereins Deutscher Portlandzement-Fabrikanten. (80) Serial beginning Aug. 21.
Temperaturerhöhung beim Abbinden von Portlandzement. (Report of Verein deutscher Portlandzement-Fabrikanten.) (80) Aug. 31.
Gründung des Kühlhauses beim Neubau des Dresdener Schlacht- und Viehhofes im Ostragehege mit Eisenbetonpfählen. Th. Gesztessy. (78) Sept. 1.
Eine Stampfmaschine für Beton-Probekörper. (51) Serial beginning Sept. 8.
Moderne Fabrikbauten in Armiertens Beton.* Hugo Gröger. (53) Serial beginning Sept. 10.
Die Probebelastung einer Bulbeisendecke (System Regierungsbaumeister Pohlmann), nebst Untersuchung des Genauigkeitsgrades einiger für die Durchbiegung von Eisenbetonbalken Angegebenen Formeln. G. Kaufmann. (78) Sept. 24.
Der Erhärtungsprozess der Kalkhaltigen Hydraulischen Bindemittel.* (80) Sept. 28.

Water Supply.

Water Service.* (Report of Comm. Amer. Ry. Eng. and M. of W. Assoc.) (85) Vol. 10, Pt. 1.
Steam-Driven Pumping Plant for Deep Wells and Boreholes. Alfred Towler, M. I. Mech. E. (Abstract of paper read before the Assoc. of Water Engrs.) (66) Aug. 31.
Hydro-Electric Development of the Connecticut River Power Company.* (27) Sept. 9.
The Flow of Water in Pipes, Culverts, and Channels.* Ernest H. Essex. (Paper read before the Inst. of County and Mun. Engrs.) (96) Sept. 10.
Water Supply and Purification. T. Aird Murray. (Paper read before Convention of Municipalities of Saskatchewan.) (96) Sept. 10.
The Hydro-Electric Power Plant at Svalgfos, Norway.* (14) Sept. 11
Irrigation Works of the United States Government. (12) Serial beginning Sept. 17.
The Construction of Additional Sand Filters at Pittsburgh.* (14) Sept. 18.
The 72-Inch Steel Pipe Line of the Brooklyn Water Supply.* (14) Sept. 18.
The Construction of the Hydro-Electric Development at Kilbourn, Wisconsin.* (14) Sept. 18; (27) Sept. 23.
The Failure of an Irrigation Dam.* Richard R. Lyman. (14) Sept. 18.
The Toronto Water Purification Works.* (14) Sept. 18.
Gagings of Small and Large Pipe on the Los Angeles Aqueduct System. J. B. Lippincott, M. Am. Soc. C. E. (13) Sept. 23.
The Recent Floods at Monterrey, N. L., Mexico.* G. R. G. Conway, M. Am. Soc. C. E. (13) Sept. 23.
Vernon, Vt., Hydroelectric Development.* C. E. Bascom. (64) Sept. 28.
Development of Devices for Cleaning the Sand of Slow Sand Filters.* E. M. Nichols. (13) Sept. 30.
Heightening the Assuan Dam across the Nile.* Frederick Hall Fowler. (13) Sept. 30.
Some Engineering Features of the Canadian Pacific Irrigation Project.* (60) Oct.
New Water Filtration Plant for Toledo, O. B. M. Baker. (60) Oct.
Extensions of the London Water Works.* H. E. Drew. (14) Oct. 2.
The Construction of the Barker Dam near Boulder.* (14) Oct. 2.
The Iron-Lime Process of Water Purification. C. Arthur Brown. (Abstract of paper read before the Central States Water Works Assoc.) (14) Oct. 9.
The Municipal Hydro-Electric Works for Winnipeg.* (14) Oct. 9.
Zur Berechnung der Staumauern.* Schäffer. (81) Vol. 5, 1909.
Die Hochbauten der Gruppenwasserwerke in der Provinz Rheinhessen.* v. Boehmer. (49) Vol. 8 and 9, 1909.

*Illustrated.

Water Supply—(Continued).

Wasserschläge in Wasserleitungen. A. Rateau. (From the "Bulletin Scientifique de l'Association des Elèves des Ecoles Spéciales," April, 1909.) (97) Serial beginning July 20.
Zur Klärung der Stosswirkung des Wassers und des Stossverlustes in den Turbinen.* L. Quantz. (97) Aug. 20.
Die Abhängigkeit des Wirkungsgrades der Wasserturbinen von Gefälle, Wasserwärme, Turbinengrösse und Rauheit der Kanäle. Dr. Camerer. (48) Sept. 18.
Beitrag zur Theorie der Zentrifugalpumpen. G. Zerkowitz. (53) Sept. 24.

Waterway.

The Water-Front of New York City. J. A. Bensel. (6) July.
Widening the American Canal at Sault Ste. Marie.* (14) Sept. 11.
The Lock Gates of the Panama Canal.* (13) Sept. 16.
The Waterways and Waterfalls of Finland.* (12) Sept. 17.
Improvements in the Navigation of the St. Lawrence. Wm. P. Anderson, Past Pres. Can. Soc. C. E. (Paper read before the Brit. Assoc.) (11) Sept. 17.
Construction Plants for the Pacific Locks of the Panama Canal.* (14) Sept. 25.
Notes on the Panama Canal, Past and Present.* H. F. Hodges. (Abstract of paper read before the Engrs. Soc. of Penn.) (13) Sept. 30.
XI^{e} Congrès International de Navigation tenu à Saint-Pétersbourg en 1908; Rapport des Délégués Français sur les Travaux du Congrès. (43) July.
Les Ports Maritimes de la Turquie, Ports Existants et Ports Projetés.* Louis Godard. (33) Sept. 4.
Einiges aus der Geschichte und dem Bau des Suezkanals. Heidegger. (Abstract of paper read before the Architekten- und Ingenieur-Verein für Niederrhein und Westfalen.) (51) June 12.
Die Neuen Kaischuppen im Stettiner Freibezirk.* (78) July 22.

*Illustrated.

PROCEEDINGS

OF THE

AMERICAN SOCIETY

OF

CIVIL ENGINEERS

(INSTITUTED 1852)

VOL. XXXV—No. 9

NOVEMBER, 1909

Edited by the Secretary, under the direction of the Committee on Publications.

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NEW YORK 1909

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ON BITUMINOUS MATERIALS FOR ROAD CONSTRUCTION: W. W. Crosby, A. W. Dean, H. K. Bishop, A. H. Blanchard.

The House of the Society is open from 9 A.M. to 10 P.M. every day, except Sundays, Fourth of July, Thanksgiving Day, and Christmas Day.

HOUSE OF THE SOCIETY—220 WEST FIFTY-SEVENTH STREET, NEW YORK.

TELEPHONE NUMBER.....5913 Columbus.
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Vol. XXXV. NOVEMBER, 1909. No. 9.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PROCEEDINGS

This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

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MINUTES OF MEETINGS OF THE SOCIETY

October 20th, 1909.—The meeting was called to order at 8.30 P. M.; A. L. Bowman, M. Am. Soc. C. E., in the chair; Chas. Warren Hunt, Secretary; and present, also 106 members and 20 guests.

A paper by Chester W. Larner, Esq., entitled "Characteristics of Modern Hydraulic Turbines," was presented by the author, and illustrated with lantern slides.

The Secretary read a communication on the subject from Lewis F. Moody, Esq., and the paper was discussed orally by Messrs. E. Kuichling, John C. Parker, W. G. Moler, and the author.

Adjourned.

November 3d, 1909.—The meeting was called to order at 8.30 P. M.; George W. Tillson, Director, Am. Soc. C. E., in the chair; T. J. McMinn, Assistant Secretary, acting as Secretary; and present, also, 105 members and 19 guests.

A paper by T. Howard Barnes, M. Am. Soc. C. E., entitled "The Reinforced Concrete Wharf of the United Fruit Company at Bocas del Toro, Panama," was presented by the author, and illustrated with lantern slides. A communication on the subject from John Hawksworth, Assoc. M. Am. Soc. C. E., was read by the Assistant Secretary.

A paper by J. A. Lahmer, M. Am. Soc. C. E., entitled "River Protection Work on the Kansas City Southern Railway near Braden, Okla.," was presented by the Assistant Secretary.

The Assistant Secretary announced the following deaths:

GEORGE THOMAS BARNSLEY, elected Junior May 31st, 1892; Associate Member, May 7th, 1894; Member, February 6th, 1906; died October 23d, 1909.

ELSTNER FISHER, elected Junior April 3d, 1889; Associate Member, June 2d, 1897; Member, March 5th, 1902; died October 12th, 1909.

CHARLES SEWALL GOWEN, elected Member March 7th, 1888; died October 19th, 1909.

ROBERT L. ENGLE, elected Member, September 7th, 1881; died October 17th, 1909.

WILLIAM PARKER, elected Member, March 7th, 1900; died September 30th, 1909.

LIBERTY GILBERT MONTONY, elected Junior, May 3d, 1892; Associate Member, February 5th, 1896; died October 20th, 1909.

Adjourned.

ELECTIONS BY THE BOARD OF DIRECTION

The following candidates for membership were elected on the ballot canvassed November 8th, 1909:

As Members.

HOWARD ELMER ARTHUR, Dallas, Tex.
FRANK CRANSTOUN BOGGS, Washington, D. C.
NATHANIEL ELLERY, Sacramento, Cal.
WILLIAM BENJAMIN GREGORY, New Orleans, La.
SCHUYLER BRUSH KNOX, New York City.
MADISON JOHNSON LORRAINE, Berkeley, Cal.
CHARLES HART NICHOLS, New York City.
CLINTON LYTER SCHERER, Beaumont, Tex.
CHESTER ASHLEY THOMAS, Dawson, Y. T., Canada.

As Associate Members.

ROGER WELLINGTON ARMSTRONG, New York City.
CHARLES DE LA PLANE ATTERBURY, Centralia, Wash.

WILFRED KEEFER BARNARD, Los Angeles, Cal.
RALPH PETERS BLACK, Charleston, W. Va.
BUTLER BENNETT BOYD, San Diego, Cal.
FRANK MILTON CARHART, Boise, Idaho.
MORGAN CILLEY, Pasig, Philippine Islands.
FRANK JOSEPH COLLINS, Syracuse, N. Y.
JOSEPH EDWIN CRAIG, Tallulah, La.
WALTER DEVERE DENT, Greenville, S. C.
DANIEL BARTHOLOMEW DONOVAN, Rome, N. Y.
JOHN DUDER, Salt Lake City, Utah.
JOHN EDWARD EVANS, Detroit, Mich.
WILLIAM SANBORN FARLEY, Berkeley, Cal.
JOSEPH CANBY HADSALL, Wheatland, Wyo.
WILLIAM ABBOTT HAZARD, Buffalo, N. Y.
ALFRED RANDOLPH JAMES, Corozal, Canal Zone, Panama.
FREDERICK ARMISTEAD JONES, Van Buren, Ark.
ENGELBERT CONRAD LAWRENCE, Newport News, Va.
CLARENCE BURTON LONG, Fort Shaw, Mont.
ALFRED BOARDMAN MAYHEW, Boise, Idaho.
GEORGE NATHAN MITCHAM, Auburn, Ala.
FREDERICK EDGAR MORROW, Chicago, Ill.
ARTHUR TRAVER NEWMAN, New York City.
THEODORE WHITE NORCROSS, Mundale, Mass.
ALBERT FRANCIS PLOCK, Mayville, Wis.
HENRI RUSCH, St. Louis, Mo.
EDWARD CHARLES STRATHMANN, Indianapolis, Ind.
WILLIAM MICHAEL THOMAS, Los Angeles, Cal.
ERIK BIRGER LUND UNGER, Mexico, D. F., Mexico.
EDWARD MANSFIELD WALKER, Detroit, Mich.
MARVIN CURTIS WELBORN, Austin, Tex.
JULIUS RALPH WEMLINGER, Brooklyn, N. Y.
DAVID MILLER WHITE, Midland, Tex.
LESTER DENNISON WILLIAMS, Milwaukee, Wis.

As Associate.

ALLERTON SEWARD CUSHMAN, Washington, D. C.

As Juniors.

HUGH JAMES BETTERLEY, Corregidor Island, Philippine Islands.
ERNEST OSWALD BILLWILLER, San José, Cal.
HERMAN WILLIAM BLUHM, Corozal, Canal Zone, Panama.
JOHN NIXON BROOKS, Trenton, N. J.
WALTER BENJAMIN BUSHWAY, Cambridge, Mass.
J. C. CARPENTER, Watertown, S. Dak.
EMERSON LAWRENCE CHANDLER, New London, Conn.
WALTER WOODBRIDGE CLIFFORD, Hyde Park, Mass.

George Edmund Dunan, Apalachicola, Fla.
Glen Edgar Edgerton, Culebra, Canal Zone, Panama.
Isaac Oberholzer Gibble, Hibbing, Minn.
Germain Paul Graham, Albany, N. Y.
Albert Paul Haney, New York City.
Charles Ellsworth Haywood, Mariner Harbor, N. Y.
Richard Hazen, Yonkers, N. Y.
Edmund Geddes King, Pottsville, Pa.
John Raymond Lapham, West Medway, Mass.
Oliver Yeaton Leonard, Guayama, Porto Rico.
Edwin Neuhardt, Memphis, Tenn.
Bill Read, Redwood City, Cal.
Walter Vanderbelt Scott, Hudson, N. Y.
Ralph Graham Shankland, Chicago, Ill.
Walter Phelps Stewart, Nashville, Tenn.
Otto Jordan Swensson, Yorktown Heights, N. Y.
Robert Lee Tatum, Rosedale, Miss.
Nathan Thomas Veatch, Jr., Kansas City, Mo.
Sydney Wilmot, Bristol, Conn.
Lawrence Lewis Winans, Mexico, Mo.

Transfers from Associate Member to Member.

Frederick William Dencer, Chicago, Ill.
William Nelson Hazen, Newark, N. J.
George Albert McKay, North Chicago, Ill.
Clyde Tucker Morris, Columbus, Ohio.
George Reed Wadsworth, Boston, Mass.
Nathaniel Alston Yuille, Mobile, Ala.

Transfer from Associate to Member.

Edward John Schneider, San Francisco, Cal.

Transfers from Junior to Associate Member.

Francis Rigdon Berry, Portsmouth, Va.
William Gomer Davies, Willows, Cal.
Oscar Hasbrouck, Cohoes, N. Y.
Jonathan Yates Higginson, New Rochelle, N. Y.
Ray Palmer Hovey, Salt Lake City, Utah.
Edgar Staples Maupin, Vicksburg, Miss.
James Henry Miner, Montrose, Colo.
Hallet Rice Robbins, Keremeos, B. C., Canada.
Forrest Leigh Smith, Perth Amboy, N. J.
Louis Bernard Spencer, Ogden, Utah.
Guy Anderson Watkins, Little Rock, Ark.
George William Carlyle Whiting, Baltimore, Md.
Robert Walter Wood, Mariner Harbor, N. Y.

OF THE BOARD OF DIRECTION

(Abstract)

November 8th, 1909.—President Bates in the chair; Chas. Warren Hunt, Secretary, and present, also, Messrs. Andrews, Benzenberg, Brackett, Christie, Churchill, Endicott, Harrison, Hazen, Hodgdon, Horton, Kittredge, Knap, Noble, Pegram, Schneider, Stuart, Swensson, Talbot, Thompson, Tillson, and Wilkins.

A communication was received from V. G. Bogue, M. Am. Soc. C. E., who was nominated by the Nominating Committee for the office of Director, representing District No. 7, stating that he expects soon to move out of that district, and therefore cannot be a candidate for that office. In accordance with the Constitutional provision, the Board filled the vacancy caused by Mr. Bogue's inability to serve by substituting Arthur DeWint Foote, M. Am. Soc. C. E., of Grass Valley, Cal.

Ballots for membership were canvassed, resulting in the election of 9 Members, 35 Associate Members, 1 Associate, and 28 Juniors, and the transfer of 6 Associate Members and 1 Associate to the grade of Member, and 13 Juniors to the grade of Associate Member.

The following resignations were accepted: C. C. English, Jun. Am. Soc. C. E., J. O. Van Brakle, Jun. Am. Soc. C. E., and Houston Lowe, Assoc. Am. Soc. C. E.

Applications were considered and other routine business transacted.

Adjourned.

ANNOUNCEMENTS

The House of the Society is open from 9 A. M. to 10 P. M., every day, except Sundays, Fourth of July, Thanksgiving Day, and Christmas Day.

MEETINGS

December 1st, 1909.—8.30 P. M.—At this meeting a paper by James H. Brace and Francis Mason, Members, Am. Soc. C. E., entitled "The New York Tunnel Extension of the Pennsylvania Railroad: The Cross-Town Tunnels," will be presented for discussion.

This paper was printed in *Proceedings* for October, 1909.

December 15th, 1909.—8.30 P. M.—At this meeting two papers will be presented for discussion as follows: "The New York Tunnel Extension of the Pennsylvania Railroad: The East River Tunnels," by James H. Brace, Francis Mason, and S. H. Woodard, Members, Am. Soc. C. E., and "The New York Tunnel Extension of the Pennsylvania Railroad: Contractors' Plant for East River Tunnels," by Henry Japp, M. Am. Soc. C. E.

The paper by Messrs. Brace, Mason, and Woodard was printed in *Proceedings* for October, 1909, and Mr. Japp's paper is printed in this number of *Proceedings*.

January 5th, 1910.—8.30 P. M.—At this meeting a paper by Horace J. Howe, M. Am. Soc. C. E., entitled "Notes on the Replacing of the Superstructure of the Harlem Ship Canal Bridge," will be presented for discussion.

This paper is printed in this number of *Proceedings*.

ANNUAL MEETING

The Fifty-seventh Annual Meeting will be held at the Society House, on Wednesday and Thursday, January 19th and 20th, 1910. The Business Meeting will be called to order at 10 o'clock on Wednesday morning. The Annual Reports will be presented, officers for the ensuing year elected, members of the Nominating Committee appointed, Reports of Special Committees presented for discussion, and other business transacted.

LIST OF NOMINEES FOR OFFICES TO BE FILLED AT THE ANNUAL ELECTION, JANUARY 19th, 1910.

In the October number of *Proceedings* the list of nominees for the offices to be filled at the Annual Meeting, January 19th, 1910, received from the Nominating Committee, was published, and a copy of this list had also previously been mailed to all Corporate Members.

Virgil G. Bogue, M. Am. Soc. C. E., now of San Francisco, Cal., subsequently notified the Board of Direction that it was his intention

to move out of District No. 7, in January next, in consequence of which he could not be a candidate for Director from that district.

The Board of Direction, under the authority vested in it by the Constitution, has substituted the name of Arthur DeWint Foote, M. Am. Soc. C. E., of Grass Valley, Cal., for that of Mr. Bogue as the nominee for Director to represent District No. 7 for the next three years.

MEETINGS OF THE SAN FRANCISCO ASSOCIATION OF MEMBERS, AM. SOC. C. E.

The San Francisco Association of Members of the American Society of Civil Engineers holds regular bi-monthly meetings, with banquet, and weekly informal luncheons. The former are held at 6 P. M., at the Fairmont Hotel, on the third Friday of February, April, June, August, October, and November, and also on the third Wednesday of December, the latter being the Annual Meeting of the Association.

Informal luncheons are held at 12.15 P. M. every Wednesday, and the place of meeting may be ascertained by communicating with the Secretary of the Association, E. T. Thurston, Jr., Assoc. M. Am. Soc. C. E., 623 Wells Fargo Building.

The by-laws of the Association provide for the extension of hospitality to any members of the Society who may be temporarily in San Francisco, and any such member will be gladly welcomed as a guest of the Association at any of the above meetings, if he will notify the Secretary that he is in San Francisco.

PAPERS AND DISCUSSIONS

The first three volumes of *Transactions* for 1909 (Volumes LXII, LXIII, and LXIV) have been issued. The fourth volume will be issued on December 31st.

It is hoped that members and others who take part in the discussion of the papers presented will revise their remarks promptly, and that all written communications from those who cannot attend the meetings will be sent in at the earliest possible date after the issue of a paper in *Proceedings*. The issue of volumes of *Transactions* is dependent on the closing of discussions, and the co-operation of the membership will now be more necessary in this matter than heretofore, because four volumes are issued during the year, instead of two, and, to accomplish this, a definite date of issue for each has to be established.

All papers accepted by the Publication Committee are classified by the Committee with respect to their availability for discussion at meetings.

Papers, which, from their general nature, appear to be of a character suitable for oral discussion, will be published as heretofore in *Proceedings*, and set down for presentation to a future meeting of the

Society, and, on these, oral discussion, as well as written communications, will be solicited.

All papers which do not come under this heading, that is to say, those which, from their mathematical or technical nature, in the opinion of the Committee, are not adapted to oral discussion, will not be scheduled for presentation to any meeting. Such papers will be published in *Proceedings* in the same manner as those which are to be presented at meetings, but written discussions, only, will be requested for subsequent publication in *Proceedings* and with the paper in the volumes of *Transactions*.

PRIVILEGES OF ENGINEERING SOCIETIES EXTENDED TO MEMBERS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

Members of the American Society of Civil Engineers will be welcomed by the following Engineering Societies, both to the use of their Reading Rooms and at all Meetings:

American Institute of Mining Engineers, 29 West Thirty-ninth Street, New York City.

Associação dos Engenheiros Civis Portuguezes, Lisbon, Portugal.

Australasian Institute of Mining Engineers, Melbourne, Victoria, Australia.

Boston Society of Civil Engineers, 715 Tremont Temple, Boston, Mass.

Brooklyn Engineers' Club, 197 Montague Street, Brooklyn, N. Y.

Canadian Society of Civil Engineers, 413 Dorchester Street, West, Montreal, Que., Canada.

Civil Engineers' Society of St. Paul, St. Paul, Minn.

Cleveland Engineering Society, 718 Caxton Building, Cleveland, Ohio.

Cleveland Institute of Engineers, Middlesbrough, England.

Colorado Association of Members, Am. Soc. C. E., H. J. Burt, Secy., 235 Equitable Building, Denver, Colo.

Engineers' and Architects' Club of Louisville, Ky., 303 Norton Building, Fourth and Jefferson Streets, Louisville, Ky.

Engineers' Club of Baltimore, Baltimore, Md.

Engineers' Club of Central Pennsylvania, Corner Second and Walnut Streets, Harrisburg, Pa.

Engineers' Club of Minneapolis, 17 South Sixth Street, Minneapolis, Minn.

Engineers' Club of Philadelphia. 1317 Spruce Street, Philadelphia, Pa.

Engineers' Club of St. Louis, 3817 Olive Street, St. Louis, Mo.

Engineers' Club of Toronto, 96 King Street, West, Toronto, Ont., Canada.

Engineers' Society of Western Pennsylvania, 803 Fulton Building, Pittsburg, Pa.

Institute of Marine Engineers, 58 Romford Road, Stratford, London, E., England.

Institution of Engineers of the River Plate, Buenos Aires, Argentine Republic.

Institution of Naval Architects, 5 Adelphi Terrace, London, W. C., England.

Junior Institution of Engineers, 39 Victoria Street, Westminster, S. W., London, England.

Koninklijk Instituut van Ingenieurs, The Hague, The Netherlands.

Louisiana Engineering Society, 604 Tulane-Newcomb Building, New Orleans, La.

Memphis Engineering Society, Memphis, Tenn.

Midland Institute of Mining, Civil and Mechanical Engineers, Sheffield, England.

Montana Society of Engineers, Butte, Montana.

North of England Institute of Mining and Mechanical Engineers, Newcastle-upon-Tyne, England.

Oesterreichischer Ingenieur- und Architekten-Verein, Eschenbachgasse 9, Vienna, Austria.

Pacific Northwest Society of Engineers, 617-618 Pioneer Building, Seattle, Wash.

Rochester Engineering Society, Rochester, N. Y.

Sachsischer Ingenieur- und Architekten-Verein, Dresden, Germany.

Sociedad Colombiana de Ingenieros, Bogota, Colombia.

Societe des Ingenieurs Civils de France, 19 Rue Blanche, Paris, France.

Society of Engineers, 17 Victoria Street, Westminster, S. W., London, England.

Svenska Teknologforeningen, Brunkebergstorg 18, Stockholm, Sweden.

Tekniske Forening, Vestre Boulevard 18–1, Copenhagen, Denmark.

Western Society of Engineers, 1737 Monadnock Block, Chicago, Ill.

SEARCHES IN THE LIBRARY

In January, 1902, the Secretary was authorized to make searches in the Library, upon request, and to charge therefor the actual cost to the Society for the extra work required. Since that time many searches have been made, and bibliographies and other information on special subjects furnished.

The resulting satisfaction, to the members, who have made use of the resources of the Society in this manner, has been expressed frequently, and leaves little doubt that, if it were generally known to the membership that such work would be undertaken, many would avail themselves of it.

The cost is trifling compared with the value of the time of an engineer who looks up such matters himself, and the work can be performed quite as well, and much more quickly, by persons familiar with the Library.

In asking that such work be undertaken, members should specify clearly the subject to be covered, and whether references to general books only are desired, or whether a complete bibliography, involving search through periodical literature, is desired.

In reference to this work, the Appendix* to the Annual Report of the Board of Direction for the year ending December 31st, 1906, contains a summary of all searches made to that date.

LOCAL ASSOCIATIONS OF MEMBERS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

(Abstract of Minutes of Meeting)

San Francisco Association

August 20th, 1909.—The twenty-sixth regular meeting of the San Francisco Association of Members, Am. Soc. C. E., was held; John D. Galloway, Vice-President, in the chair; M. C. Couchot, Secretary *pro tem.;* and present, also, 32 members and guests.

The recent failure of the hydraulic-fill dam at Necaxa, Mexico, was discussed by Messrs. Adams, Wise, Galloway, Hyde, Wadsworth, and Tower.

The attention of Members of the Association was called to the relief map of the Panama Canal Zone on exhibition in the rooms of the Chamber of Commerce.

Adjourned.

Colorado Association

September 11th, 1909.—The regular meeting of the Colorado Association of Members, Am. Soc. C. E., was called to order at 8 P. M.; H. S. Crocker, President, in the chair; H. J. Burt, Secretary; and present, also, 14 members and 2 guests.

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* *Proceedings*, Vol. XXXIII, p. 20 (January, 1907).

The paper by John C. Hoyt, M. Am. Soc. C. E., on "The Use and Care of the Current Meter, as Practiced by the United States Geological Survey," was discussed by Messrs. D. G. Thomas, Walter Pearl, and Gerard H. Matthes.

The paper by James H. Fuertes, M. Am. Soc. C. E., on "The Purification of the Water Supply of Steelton, Pennsylvania," was discussed by Messrs. George T. Prince, D. G. Thomas, Gavin N. Houston, and Joseph A. Sargent.

Adjourned.

October 9th, 1909.—The regular meeting of the Colorado Association of Members, Am. Soc. C. E., was held at the Platte Cañon Filter Beds, by invitation of The Denver Union Water Company.

The members and their guests left Denver at 2.30 P. M. by special train, and spent the afternoon visiting the various points of interest at the plant, supper being served at 6 P. M.

The meeting was called to order at 8 P. M., H. J. Burt, Secretary-Treasurer, in the chair; Mrs. D. G. Thomas, Secretary *pro tem.;* and present, also, 21 members and 48 guests.

A paper describing the Slow Sand Filter Plant at Platte Cañon was presented by George T. Prince, M. Am. Soc. C. E.

J. E. Maloney, M. Am. Soc. C. E., presented a paper on the Operation of the Filter Plant.

After a few remarks by Mrs. Friest, one of the visiting ladies, D. G. Thomas, M. Am. Soc. C. E., described the Distribution of Water in the City of Denver.

By special request, Mr. DeBrerard, Chemist for The Denver Union Water Company, explained the chemical and bacteriological qualities of the water, exhibiting samples in different degrees of turbidity.

Mr. E. C. Van Diest, of Colorado Springs, Colo., gave a brief description of the water supply of that city.

A resolution of thanks to The Denver Union Water Company, for the courtesies, entertainment, and interesting descriptions of the Filter Plant, was adopted unanimously.

Adjourned.

ACCESSIONS TO THE LIBRARY

(From October 12th to November 8th, 1909)

DONATIONS*

A TREATISE ON CONCRETE PLAIN AND REINFORCED.

Materials, Construction, and Design of Concrete and Reinforced Concrete, with Chapters by R. Feret, William B. Fuller, Frank P. McKibben, and Spencer B. Newberry. By Frederick W. Taylor and Sanford E. Thompson, M. Am. Soc. C. E. Second Edition. Cloth, 9 x 6 in., illus., 18 + 807 pp. New York, John Wiley & Sons; London, Chapman & Hall, Limited, 1909. $5.00. (Donated by Sanford E. Thompson, M. Am. Soc. C. E.)

The preface to the first edition of this work, published in 1905, states that each chapter has been submitted to at least one, and, in some cases, three or four specialists in the particular line treated. In the present edition, the aim, the authors state, has been to cover the developments in the design and construction of reinforced concrete since the first issue of the work. To accomplish this, more than two hundred pages of entirely new text and tables have been added, giving, the authors claim, to the constructing engineer, the architect, and the contractor, data for design and for building, and to the student a comprehensive and practical text and reference book. The authors direct special attention to the new Chapter I, in which many of the essentials of concrete construction are pointed out, and the reader warned against the serious errors frequently made in this field; to Chapter III, which contains the Revised Specifications for Cement and Concrete; to Chapter IX, on Proportioning; to Chapters XIV and XV, on Mixing and Depositing, and to the greatly enlarged Chapter on Reinforced Concrete Design. The list of references in Chapter XXXI has been increased, it is stated, more than 50%, new references having been carefully selected from the current literature published since the issue of the first edition.

A BRIEF HISTORY OF CEMENTS.

By I. C. Johnson. Paper, 9 x 6 in., illus., 6 + 27 pp. Kansas City, Mo., The Cement Record Co., 1909. 50 cents.

The author claims to be the inventor and pioneer of the Portland cement industry, and describes the process of its evolution—how it was invented, how developed, and how first manufactured.

THE CIVIL ENGINEER'S POCKET-BOOK.

By John C. Trautwine. Revised by John C. Trautwine, Jr., Assoc. Am. Soc. C. E., and John C. Trautwine, 3d. Nineteenth Edition. Morocco, 6¾ x 4 in., illus., 1 300 pp. New York, John Wiley & Sons; London, Chapman & Hall, Limited, 1909. $5.00 net. (Donated by the Revisers.)

The first edition of this work, of which the present edition is the nineteenth, appeared in 1872 and the eighteenth edition in 1902. The preface to the present edition states that it has been extensively revised, and that about 300 pages of new material have been added. It further states the most notable of the new features to be the series of articles on concrete, plain and reinforced, including cement, sand, and mortar. The revisers characterize all this matter as entirely new, as far as the Pocket Book is concerned, though by no means original, their object being to present, in convenient and condensed form, the essentials of existing practice and opinion on these subjects. The articles "Selected Results of Experiment and Practice" and "Digest of Specifications" are especially mentioned as containing, in the revisers' opinion, a more complete and more conveniently classified presentation of modern concrete practice than is to be found elsewhere in equal space. This edition also contains the table of six-place "Logarithmic Trigonometric Functions" by William H. Searles, M. Am. Soc. C. E., and the diagrams representing the principal results as to flow of water in pipes and conduits given in "Hydraulic Tables," by Gardner S. Williams and Allen Hazen, Members, Am. Soc. C. E. The Price List, the Business Directory, and the Bibliography are stated to have been thoroughly revised and brought as nearly up to date as practicable in a work of this character. The glossary of engineering terms has been dropped. The Contents

* Unless otherwise specified, books in this list have been donated by the publisher.

are: Mathematics; Natural Phenomena; Mechanics, Force in Rigid Bodies; Strength of Materials; Hydrostatics; Hydraulics; Constructions, etc.; Water Supply; Test and Well Boring; Rock Drills; Traction, Animal Power; Trusses; Suspension Bridges; Rivets and Riveting; Railroads; Materials; Price List, etc.; Bibliography; Logarithmic Sines, etc.; Concrete; Index.

CONCRETE.

By John C. Trautwine, Jr., Assoc. Am. Soc. C. E., and John C. Trautwine, 3d. Reprinted from Trautwine's "Civil Engineer's Pocket-Book," Nineteenth Edition. Cloth, 7½ x 6¾ in., illus., 7 + 188 pp. New York, John Wiley & Sons; London, Chapman & Hall, Limited, 1909. $2.00 net. (Donated by the Authors.)

WHEN RAILROADS WERE NEW.

By Charles Frederick Carter. With Introductory Note by Logan G. McPherson. Cloth, 8¾ x 5½ in., illus., 12 + 324 pp. New York, Henry Holt & Company; London, George Bell & Sons, 1909. $2.00 net.

In this volume an attempt has been made, states the author's preface, to gather the floating fragments of railroad history, having a human interest, into a coherent narrative of the work-a-day trials of the pioneers in the planning and building of the railroad, which will be neither a dry historical treatise nor a collection of anecdotes. The author expresses the hope that, although the subject-matter is not comprehensive in the sense of including details of all the early railroads, or even of all the important ones which have survived, it may be found to be sufficiently comprehensive to present a picture of the development of the railroad in America under various representative types of conditions. Much of the material has been published in a series of articles in the *Railroad Man's Magazine*. The Contents are: Introduction; The Dawn of the Railroad Era; America's Pioneer Railroad; Early Days on the Erie; Pennsylvania and the Pennsylvania Railroad; Genesis of the Vanderbilt System; Incubator Railroads; The First Transcontinental Railroad; Through Tribulation by Rail; Romance of a Great Railroad; Index.

LIGHTING ENGINEERS' HAND-BOOK.

Compiled by L. R. Pomeroy, Assoc. Am. Soc. C. E. Morocco, 7 x 4¼ in., illus., 231 pp. New York, The Safety Car Heating and Lighting Co., 1909. $1.00. (Donated by the Compiler.)

The foreword characterizes this book as "extracts from the Compiler's note-book." The Contents are: General; Electricity; Lighting; Steam; Heat and Heating; Traction; Pipes and Tubes; Hydraulic; Index.

FOWLER'S MECHANICS' AND MACHINISTS' POCKET-BOOK AND DIARY, 1910.

A Synopsis of Practical Rules for Fitters, Turners, Millwrights, Erectors, Pattern Makers, Foundrymen, Draughtsmen, Apprentices, Students, etc. Edited by William H. Fowler. Second Edition. Cloth, 6½ x 3½ in., illus., 95 + 448 pp. Manchester, England, Scientific Publishing Co., 1910. 6 pence net.

The contents of the present edition of this work are said, by the editor, to have been revised and amended so as to bring it abreast with the latest practice, and a number of additional tables and new data have been added. The Contents are: Index; Handy References and Tables; Mensuration, Geometry, and Trigonometry; Uses of Logarithms and Antilogarithms; Materials Used in Machine Construction; Machine Tool Design; Proportions of Machine Tool Parts; Metal Cutting Tools; High Speed Tool Steels; Drilling and Boring Metal; Screw Threads, Screw Cutting, and Taper Turning; Emery and Emery Wheels; Shop Practice; Wheel Gearing; Belt and Rope Driving, Shafting; Lifting Ropes and Chains.

LO ZINCO.

By Professor Roberto Musu-Boy. Boards, 6 x 4 in., illus., 14 + 219 pp. Milano, Ulrico Hoepli, 1909. 3 lira 50 centimes.

In his preface the author states that, as a manual cannot, from its very nature, be a broad scientific treatise, he has, after collecting the most recent data, condensed the material as much as possible, and referred the reader who desires further explanation or a broad scientific treatment of the subject, to the authors cited

in the notes. He states further that he has divided the subject into five parts as follows: Scientific, giving the history of the origin, formation, and constitution of zinc and its ores; Industrial, telling how zinc is recovered from the ores, etc.; Mining; Practice, explaining the uses of zinc, and giving economic and statistical data. To these is added an appendix on the manufacture of zinc-white.

Gifts have also been received from the following:

Am. Academy of Political and Social Sci. 1 pam.
Am. Inst. of Elec. Engrs. 2 bound vol.
Am. Street and Interurban Ry. Accountants Assoc. 2 vol.
Am. Street and Interurban Ry. Assoc. 2 bound vol.
Architectural Inst. of Canada. 1 pam.
Assoc. of Licensed Automobile Mfrs. 1 pam.
Assoc. of Ontario Land Surv. 1 pam.
Assoc. of Vermont Engrs. 1 pam.
Baltimore & Ohio R. R. Co. 1 pam.
Bangor & Aroostook R. R. Co. 1 pam.
Beer, William. 4 pam.
Boston, Mass.-Dept. of Parks. 1 pam.
Brockton, Mass.-City Engr. 2 pam.
Brown, C. F. 1 pam.
Campbell, H. A. 11 pam., 1 bound vol.
Cape Soc. of Civ. Engrs. 1 pam.
Central Vermont Ry. Co. 1 pam.
Chicago & Eastern R. R. Co. 1 pam.
Chicago, Burlington & Quincy R. R. Co. 1 pam.
Chicago, Rock Island & Pacific Ry. Co. 2 pam.
Clarke, D. D. 1 bound vol.
Clarke, Eliot C. 1 bound vol.
Cleveland, Ohio-Chf. Engr. of Public Works. 8 bound vol.
Coorg, India-Public Works Dept. 1 pam.
Dawson, W. Bell. 4 pam.
East Orange, N. J.-Shade Tree Comm. 5 pam.
Eichel, Eugen. 1 pam.
Evansville & Terre Haute R. R. Co. 1 pam.
Farley, G. P. 1 bound vol.
Fickes, E. S. 1 pam.
Fisk & Robinson. 1 pam.
Great Northern Ry. Co. 1 pam.
Greenman, A. A. 1 pam.
Gregory, Alfred C. 1 pam.
Humphrey, R. L. 1 vol.
Illinois, Univ. of. 2 vol., 2 pam.
Illinois Univ.-Eng. Exper. Station. 2 pam.
Indiana Eng. Soc. 1 pam.
Institution of Elec. Engrs. 1 pam.
Iowa-Board of R. R. Commrs. 1 bound vol.
Iowa-State Board of Health. 1 bound vol.
Iowa State Coll. of Agriculture and Mechanic Arts. 2 vol.
Iowa State Univ. 2 vol.
Jacksonville, Fla.-Board of Trustees for the Waterworks and Impvt. Bonds. 1 pam.
Junior Institution of Engrs. 1 vol.
Lewis, Nelson P. 1 bound vol.
Louisville & Nashville R. R. Co. 1 pam.
Lynn, Mass.-City Messenger. 1 bound vol.
McMinn, T. J. 1 vol., 2 pam.
Madras, India-Public Works Dept. 1 bound vol.
Manchester, England-Rivers Dept. 1 pam.
Marsh, Benjamin C. 1 pam.
Mass. Inst. of Tech. 1 pam.
Memphis, Tenn.-Board of Health. 1 pam.
Merchants Assoc. of New York. 1 pam.
Mexican Inter. R. R. Co. 1 pam.
Mexican Ry. Co., Limited. 4 pam.
Montreal, Canada - Medical Officer of Health. 1 pam.
Morris, Marshall. 1 bound vol.
National Board of Fire Underwriters. 13 pam.
New South Wales-Govt. Statistician. 1 bound vol.
New York City-Dept. of Finance. 1 pam.
New York City-Board of Water Supply. 1 pam.
New York State-Commr. of Excise. 1 bound vol.
New York-State Engr. and Surv. 1 map.
New York State-Public Service Comm. 1 pam.
Northern Pacific Ry. Co. 1 pam.
Ockerson, J. A. 1 pam.
Ohio Eng. Soc. 1 pam.
Ontario-Provincial Board of Health. 1 pam.
Oesterreichischer Ingenieur- und Architekten-Verein. 2 pam.
Outdoor Art League. 1 pam.
Page, William N. 1 pam.
Philadelphia Rapid Transit Co. 7 pam.
Philippine Islands-Bureau of Sci. 1 pam.
Quebec Central Ry. Co. 1 pam.
Queensland-Commr. of Rys. 1 pam.
Reading Co. 1 pam.
Rensselaer Soc. of Engrs. 1 bound vol.
Rousseau, H. H. 1 pam.
Royal Soc. of Canada. 1 vol.
St. Louis Southwestern Ry. Co. 1 pam.
São Paulo-Commissão Geographica e Geologica. 1 map.
Sayward, William H. 1 pam.
Stearns, Frederic P. 2 pam.
Tarrytown, N. Y.-Board of Water Commrs. 2 pam.
Toledo, St. Louis & Western R. R. Co. 1 pam.
Tulane Univ. of Louisiana. 1 pam.
Union Pacific R. R. Co. 1 pam.
U. S.-Geol. Survey. 23 pam., 38 maps.
U. S.-Isthmian Canal Comm. 1 bound vol.
U. S.-Lake Survey Office. 1 pam.
U. S.-Office of Exper. Stations. 2 pam.
U. S.-National Museum. 1 pam., 1 bound vol.
Verband Deutscher Diplom-Ingenieure. 2 pam.
Verein Deutscher Portland-Zement Fabrikanten. 1 vol.
Woodworth, R. B. 1 pam.

BY PURCHASE

Leitfaden zum Berechnen und Entwerfen von Lüftungs- und Heizungs-Anlagen, ein Hand- und Lehrbuch für Ingenieure und Architekten. By H. Rietschel. 2 vol. Julius Springer, Berlin, 1909.

Manufacture of Explosives, Twenty Years' Progress; Four Cantor Lectures Delivered at the Royal Society of Arts in November and December, 1908. By Oscar Guttmann. Whittaker & Co., London and New York, 1909.

Beiträge zur Theorie und Berechnung der vollwandigen Bogenträger ohne Scheitelgelenk, Insbesondere der Brückengewölbe und der im Eisenbetonbau Ueblichen Biegungsfesten Rahmen. By Max Ritter. Wilhelm Ernst & Sohn, Berlin, 1909.

SUMMARY OF ACCESSIONS

(From October 12th to November 8th, 1909)

Donations (including 21 duplicates)	254
By purchase	4
Total	258

MEMBERSHIP

ADDITIONS

(October 12th to November 9th, 1909)

MEMBERS		Date of Membership.
BADENHAUSEN, JOHN PHILLIPS. Contr. Engr. (Badenhausen & Hull, Inc.), 1530 Thirty-sixth Ave., Seattle, Wash.		Oct. 5, 1909
BURGESS, HARRY. Maj., Corps of Engrs., U. S. A., 3338 St. Charles Ave., New Orleans, La		Oct. 5, 1909
CONARD, CLARENCE KNIGHT. Asst. Engr., Erie R. R. Co., 50 Church St., Room 977, New York City		Oct. 5, 1909
DURHAM, HENRY WELLES. Res. Engr., Cape Cod Constr. Co., Sandwich, Mass		Oct. 5, 1909
FETHERSTON, JOHN TURNEY. Supt. of Street Cleaning, Borough of Richmond, 78 Westervelt Ave., New Brighton, N. Y.	Assoc. M. M.	June 3, 1903 Oct. 5, 1909
HAMMATT, WILLIAM CUSHING. Chf. Engr., Miller & Lux, Inc., and San Joaquin & Kings River Canal & Irrig. Co., Inc., 1313 Merchants Exchange, San Francisco, Cal	Assoc. M. M.	Sept. 5, 1900 Oct. 5, 1909
HOMBERGER, HEINRICH. Cons. Engr., Room 865, Pacific Bldg., San Francisco, Cal		Oct. 5, 1909
HOPKINS, STEPHEN UPSHUR. 620 Lumbermen's Bldg., Portland, Ore		Oct. 5, 1909
LAWRENCE, EDGAR HEISLER. Care, Eames & Young, Archts., 1802 Wright Bldg., St. Louis, Mo		Oct. 5, 1909
LINDSEY, JOHN BROWN, JR. Supt., West Pascagoula Creosoting Works, Gautier, Miss.	Assoc. M. M.	June 3, 1903 Oct. 5, 1909
MCDONNELL, ROBERT EMMETT. (Burns & McDonnell, Civ., Hydr. and San. Engrs.), Room 821, Scarritt Bldg., Kansas City, Mo		Oct. 5, 1909
MCMILLAN, JOHN GILMORE. County Surv., Santa Clara Co., San José, Cal		Oct. 5, 1909
MEANS, THOMAS HERBERT. Project Engr., U. S. Reclamation Service, Fallon, Nev	Assoc. M. M.	July 10, 1907 Oct. 5, 1909
MOORE, CHARLES GILLINGHAM. 744 Ellicott Sq., Buffalo, N. Y	Assoc. M. M.	Mar. 5, 1902 June 1, 1909
PIERCE, FREDERIC EMERY. Engr. of Constr., The New Jersey Zinc Co., 55 Wall St., New York City		Oct. 5, 1909
PITMAN, FREDERICK LONGFELLOW. Chf. Engr., The North Coast R. R. Co., 918 Paulsen Bldg., Spokane, Wash.		Oct. 5, 1909
PRESTON, CHARLES HENRY. Archt. and Structural Engr., 43 Broadway, Norwich, Conn		Oct. 5, 1909
SCHNEEWEISS, ADOLPH EUGENE. Engr., John W. Ferguson Co., 540 East 27th St., Paterson, N. J		Oct. 5, 1909

MEMBERS (*Continued*).		Date of Membership.
SONDEREGGER, ARTHUR LUDWIG. Cons. Hydr. Engr., 634 Central Bldg., Los Angeles, Cal.		Oct. 5, 1909
VAN FRANK, PHILIP RILEY, JR. Prin. Asst. Engr., U. S. Engr. Office, Little Rock, Ark.		Aug. 31, 1909
WALTER, RAYMOND F. Project Engr., U. S. Reclamation Service, Belle Fourche, S. Dak.		Oct. 5, 1909
WILSON, PERCY HARTSHORNE. Cons. Engr., 1330 Land Title Bldg., Philadelphia, Pa.	Jun. Assoc. M. M.	Jan. 2, 1900 Feb. 5, 1902 Oct. 5, 1909
ZINN, AARON STANTON. Res. Engr., Isthmian Canal Comm., Empire, Canal Zone, Panama.		Oct. 5, 1909

ASSOCIATE MEMBERS

BALCH, LELAND RELLA. Care, U. S. Reclamation Service, Neillsville, Wis.		Oct. 5, 1909
BLACK, GURDON GILMORE. First Asst. Engr., Supply and Purifying Div., St. Louis Water Dept., 34 E. Grand Ave., St. Louis, Mo.		Oct. 5, 1909
BREWER, JESSE IRVING. Asst. Engr., Interborough Rapid Transit Co., 32 Park Pl., New York City.		Oct. 5, 1909
CLAYBAUGH, HARRY WRAY. State Highway Dept. of Pa., Franklin, Pa.		June 1, 1909
CLINTON, SAMUEL DEXTER. Chf. Engr., Portneuf Marsh Valley Irrigation Co., Downey, Idaho.		Oct. 5, 1909
COPLEY, GEORGE NOBLE. Treas., Goldhart & Bates; Secy. and Treas., U. S. Eng. Co., 324 Security Bldg., Galveston, Tex.	Jun. Assoc. M.	April 3, 1906 Oct. 5, 1909
CURTIS, CLINTON ALONZO. Asst. Engr., N. Y. State Barge Canal, Ft. Edward, N. Y.	Jun. Assoc. M.	Oct. 1, 1907 Oct. 5, 1909
ENGLE, CHARLES ALGERNON. Res. Engr., Salmon River Project, Box Y, Twin Falls, Idaho.	Jun. Assoc. M.	May 5, 1908 Oct. 5, 1909
FINEREN, WILLIAM WARRICK. Junior Engr., U. S. Engr. Office, Federal Bldg., Tampa, Fla.		Oct. 5, 1909
HALDEMAN, WALTER STANLEY. 1105 Diamond St., Philadelphia, Pa.		Oct. 5, 1909
HAMLIN, RALPH. Asst. Engr., Corrugated Bar Co., Room 711, Pioneer Press Bldg., St. Paul, Minn.	Jun. Assoc. M.	Jan. 5, 1904 Oct. 5, 1909
HANSELL, WILLIAM ALBERT. Civ. Engr. and Supt. of Constr., 630 Highland Ave., Atlanta, Ga.		Oct. 5, 1909
HOLLEY, HARRY HALL. (Holley & Holley, Cons. and Superv. Engrs.), 14 First National Bank, Visalia, Cal.		Oct. 5, 1909
MIRICK, ALFRED STOWE. 144 Jay St., Albany, N. Y.		Oct. 5, 1909
MONTGOMERY, ERNEST. Box K, Palisades, Colo.		Oct. 5, 1909

ASSOCIATE MEMBERS (*Continued*).

	Date of Membership.
MOOREHEAD, THEODORE PARKER. P. O. Box 1129, El Paso, Tex	Oct. 5, 1909
RANSON, BIRTRAM WILLARD. Engr., The General Fireproofing Co., 802 Michigan Ave., Youngstown, Ohio — Jun.	Sept. 3, 1907
RANSON, BIRTRAM WILLARD. Engr., The General Fireproofing Co., 802 Michigan Ave., Youngstown, Ohio — Assoc. M.	Oct. 5, 1909
SAMPSON, CORNELIUS BRAMHALL. Conejos, Colo	Oct. 5, 1909
SHONK, JOHN JENKS. 1002 W. Broad St., Bethlehem, Pa	Oct. 5, 1909
SPLITSTONE, CHARLES HAROLD. Res. Engr., Erie R. R., 9 Girard Ave., East Orange, N. J. — Jun.	Sept. 5, 1909
SPLITSTONE, CHARLES HAROLD. Res. Engr., Erie R. R., 9 Girard Ave., East Orange, N. J. — Assoc. M.	Oct. 5, 1909
WIGGINS, RALPH RAYMOND. Asst. Engr., Chf. Engr.'s Office, Isthmian Canal Comm., Gatun, Canal Zone, Panama	Oct. 5, 1909
WONSON, SAMUEL LAMSON. Asst. Engr., Mexican Ry. Co., City of Mexico, D. F., Mexico	Oct. 5, 1909

JUNIORS

CALDWELL, JOHN WORDE. Care, U. S. Engr. Office, McCandless Bldg., Honolulu, Hawaii	Aug. 31, 1909
CHAFETZ, HERMAN. 80 Pratt St., Buffalo, N. Y	June 1, 1909
EMORY, LLOYD TILGHMAN. Care, Kaolin Min. Co., McIntyre, Ga	Oct. 5, 1909
FERGUSON, LEWIS REPP. Prof. of Mathematics, Temple Univ.; Cons. Engr., 1330 Land Title Bldg., Philadelphia, Pa	Aug. 31, 1909
HASTINGS, HUDSON BRIDGE. Asst. Prof., Dept. of Surveying and Drawing, Bowdoin Coll., Brunswick, Me	Oct. 5, 1909
HOWE, CLARENCE DECATUR. Prof. of Civ. Eng., Dalhousie Coll., Halifax, N. S., Canada	Oct. 5, 1909
HOWSON, GEORGE WILLIAM, JR. 1801 Fairview St., S. Berkeley, Cal	Oct. 5, 1909
MACFARLANE, ARTHUR KEDDIE. Care, The Anderson, Benicia, Cal	Oct. 5, 1909
MALONEY, WALDEN LE ROY. Asst. to City Engr., Bridge Dept., P. O. Box 1078, Spokane, Wash	Aug. 31, 1909
TODD, OLIVER JULIAN. 1112 Ellis St., San Francisco, Cal.	Aug. 31, 1909
TOLL, ROGER WOLCOTT. Eng. Dept., Denver City Tramway Co., 215 E. 11th Ave., Denver, Colo	Oct. 5, 1909
WESTON, FREDERICK SAMPSON. Middleboro, Mass	Oct. 5, 1909

CHANGES OF ADDRESS

MEMBERS

AERTSEN, GUILLIAEM. Mgr., Latrobe Plant, Railway Steel-Spring Co., Latrobe, Pa.

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MEMBERS (*Continued*).

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FAUNTLEROY, JAMES DEARING. Lynch Station, Campbell Co., Va.

FENN, ROBERT WILSON. Care, Union Oil Co. of California, Security Bldg., Los Angeles, Cal.

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HONENS, FREDERICK WILLIAM. U. S. Asst. Engr., U. S. Engr. Office, Postal Telegraph Bldg., Kansas City, Mo.

HOWE, HORACE JOSEPH. Asst. Engr., Public Service Comm., Room 1905 Tribune Bldg., New York City.

JANES, GEORGE PORTLOCK. 405 Third Ave. East, Roselle, N. J.

JOHNSON, LUCIEN SAMUEL. U. S. Engr. Office, Lock No. 13, Thirteen, Lee Co., Ky.

KNAPP, ZAC ELLIS. Asst. Engr., Underground Elec. Railways Co. of London, Ltd., Electric Railway House, Broadway, Westminster, London, S. W., England.

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MCFETRIDGE, WILLIAM SUTTON. Plum St., Greenville, Pa.

MCREYNOLDS, ORVAL OMAR. Cons. Civ. and Min. Engr., 318 H. W. Hellman Bldg., Los Angeles, Cal.

MACLAY, WILLIAM WALTER. Cons. Engr., 102 Erskine St., Detroit, Mich.

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NOBLE, ALFRED (*Past-President*). 7 East 42d St., New York City.

PAGE, WILLIAM NELSON. 1863 Kalorama Rd., Washington, D. C.

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MEMBERS (*Continued*).

RAYMOND, CHARLES WARD. 816½ K St., Sacramento, Cal.

RIPLEY, JOHN WESLEY. Secy. and Treas., The Robbins-Ripley Co., Room 375, 50 Church St., New York City.

ROBBINS, ALLAN APPLETON. Pres., The Robbins-Ripley Co., 50 Church St., New York City.

STEVENS, HORACE EDWARD. Director, Winston Bros. Co., Ry. Contrs., 530 Grand Ave., St. Paul, Minn.

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TAYLOR, EDWIN ALEXANDER. Supt. of Constr., Water Board, 915 East Salmon St., Portland, Ore.

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TWYMAN, AARON. West Holt, Teignmouth, Devon, England.

WALSH, GEORGE SCHERZER. Room 308, Richard Blk., Lincoln, Nebr.

WARRINGTON, HARRY ESMOND. Pepper St., near 8th St., Riverside, Cal.

WILLIAMS, CHARLES PAGE. Engr., U. S. Reclamation Service, Malta, Mont.

WOOD, WARREN POWELL. Chf. Engr., Central Idaho Development Co., Ltd., Lewiston & Southeastern Elec. Ry. Co., Ltd., Salmon River Power Co., Ltd., Elec. Ry. Townsite Co., and Nezperce & Idaho R. R. Co., Box 38, Lewiston, Idaho.

YATES, PRESTON KING. Chf. Engr., Tomkins Cove Stone Co., Tomkins Cove, N. Y.

ASSOCIATE MEMBERS

ACKENHEIL, ALFRED CURTIS. 3423 Webster Ave., Pittsburg, Pa.

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ELY, JOHN ANDREWS. Structural Engr., 90 Morningside Ave., W., New York City.

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WALTMAN, WILLIAM DEWITT. Acting Asst. Engr., Costilla Estates Development Co., Colorado Springs, Colo.

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LIEBMANN, ALFRED. 57 West 58th St., New York City.

JUNIORS

BALDRIDGE, JAMES RAMSEY. 203 Washington Park, Brooklyn, N. Y.

BERRY, FRANCIS RIGDON. Care, Portsmouth, Berkley & Suffolk Water Co., Portsmouth, Va.

BULLEN, ROY. Fifth East Hotel, Salt Lake City, Utah.

CARTER, LESTER LEVI. Box 665, Fresno, Cal.

COPE, ERLE LONG. 2316 Bowditch St., Berkeley, Cal.

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FORSYTH, HAROLD FREDERICK. Highway Commr.'s Office, Olympia, Wash.

GERMER, WILHELM EDUARD. Hennebique Constr. Co., 1170 Broadway (Res., 352 West 22nd St.), New York City.

GREEN, CLARENCE JASPER. 239 S. 12th St., Ann Arbor, Mich.

HALL, JULIUS REED. Care, Purdy & Hender, 902 Ft. Dearborn Bldg., Chicago, Ill.

JOHNSTON, ANDREW CRAWFORD. East Walpole, Mass.

JONES, GEORGE MORTON. Asst. Chf. Engr., Augusta Constr. Co., P. O. Box 223, Augusta, Ga.

KILKENNY, TOBIAS DILLON. 538 Georgia St., Vallejo, Cal.

LEWIS, CHESTER BROOKS. 1548 First National Bank Bldg., Chicago, Ill.

LORD, HAROLD. Supt. of Constr., U. S. Light-House Establishment, Office of Light-House Engr., Custom House, Portland, Ore.

LYDSTON, WALTER EDWARD. Asst. Engr., Third Ave. R. R., 130th St. and Third Ave., New York City.

MALSBURY, OMNER EVERT. Care, P. R. R. Land Office, Ancon, Canal Zone, Panama.

MILLS, ADELBERT PHILO. Asst. Prof. of Civ. Eng., Coll. of Civ. Eng., Cornell Univ., 407 N. Aurora St., Ithaca, N. Y.

NICOLAYSEN, ALBIN GEORG. 876 Irvine St., Bronx, New York City

PAGON, WILLIAM WATTERS. 1301 St. Paul St., Baltimore, Md.

PIERCE, PAUL LEON. Care, Grand Central Hotel, Valleyfield, Que., Canada.

RIEGEL, ROSS MILTON. 275 Briggs St., Harrisburg, Pa.

SCHOLTZ, HERMAN FRED. Engr. and Contr., The Gordon, 12th and Pine Sts., Seattle, Wash.

JUNIORS (*Continued*).

SISSON, GEORGE ARTHUR. Fort Stevens, Ore.

SMITH, ROBERT MACKINLAY. 3 Westbourne Terrace, Kelvinside, Glasgow, Scotland.

STEEGMULLER, CHARLES ALBERT AUGUSTINE. 26 East 198th St., Bronx, New York City.

TALLMAN, PAUL BERTRAM. Care, Warren & Wetmore, Archts., Texas and Crawford Sts., Houston, Tex.

TAYLOR, WARREN CROSBY. Care, Trussed Concrete Steel Co., Detroit, Mich.

TYLER, WILLIAM ROGERS. 164 South St., Morristown, N. J.

WELLER, WILLIAM EARL. 63 Robinson St., Schenectady, N. Y.

WHITBECK, LEE FIELD. Ingro. Residente, Ferrocarril Interoceanico de Mexico, Otumba, Mexico.

WHITSIT, LYLE ANTRIM. Care, St. Lawrence River Power Co., Massena, N. Y.

WILDER, ALVIN DUMOND. N. W. Cor. Dwight Way and Prospect, Berkeley, Cal.

WILLARD, WILLIAM CLYDE. Prof. of Railway and Highway Eng., Washington State Coll., Pullman, Wash.

WILSON, SAMUEL. 4120 Kingman Blvd., Des Moines, Iowa.

WILSON, WILLIAM WEST. 203 Washington Park, Brooklyn, N. Y.

RESIGNATION

JUNIOR	Date of Resignation.
VAN BRAKLE, JOHN OGDEN	October 5, 1909

DEATHS

BARNSLEY, GEORGE THOMAS. Elected Junior, May 31st, 1892; Associate Member, May 7th, 1894; Member, February 6th, 1906; died October 23d, 1909.

ENGLE, ROBERT L. Elected Member, September 7th, 1881; died October 17th, 1909.

FISHER, ELSTNER. Elected Junior, April 3d, 1889; Associate Member, June 2d, 1897; Member, March 5th, 1902; died October 12th, 1909.

GOWEN, CHARLES SEWALL. Elected Member, March 7th, 1888; died October 19th, 1909.

HILTON, GEORGE PORTER. Elected Member, May 1st, 1889; died October 7th, 1909.

MONTONY, LIBERTY GILBERT. Elected Junior, May 3d, 1892; Associate Member, February 5th, 1896; died October 20th, 1909.

RIFFLE, ALBERT STANLEY. Elected Member, May 7th, 1890; died October 28th, 1909.

Total Membership of the Society, November 9th, 1909, 5 198.

MONTHLY LIST OF RECENT ENGINEERING ARTICLES OF INTEREST

(October 12th to November 8th, 1909)

NOTE.—*This list is published for the purpose of placing before the members of the Society, the titles of current engineering articles, which can be referred to in any available engineering library, or can be procured by addressing the publication directly, the address and price being given wherever possible.*

LIST OF PUBLICATIONS

In the subjoined list of articles, references are given by the number prefixed to each journal in this list:

(1) *Journal*, Assoc. Eng. Soc., 31 Milk St., Boston, Mass., 30c.
(2) *Proceedings*, Engrs. Club of Phila., 1317 Spruce St., Philadelphia, Pa.
(3) *Journal*, Franklin Inst., Philadelphia, Pa., 50c.
(4) *Journal*, Western Soc. of Engrs., Monadnock Blk., Chicago, Ill.
(5) *Transactions*, Can. Soc. C. E., Montreal, Que., Canada.
(6) *School of Mines Quarterly*, Columbia Univ., New York City, 50c.
(7) *Technology Quarterly*, Mass. Inst. Tech., Boston, Mass., 75c.
(8) *Stevens Institute Indicator*, Stevens Inst., Hoboken, N. J., 50c.
(9) *Engineering Magazine*, New York City, 25c.
(10) *Cassier's Magazine*, New York City, 25c.
(11) *Engineering* (London), W. H. Wiley, New York City, 25c.
(12) *The Engineer* (London), International News Co., New York City, 35c.
(13) *Engineering News*, New York City, 15c.
(14) *The Engineering Record*, New York City, 12c.
(15) *Railroad Age Gazette*, New York City, 15c.
(16) *Engineering and Mining Journal*, New York City, 15c.
(17) *Electric Railway Journal*, New York City, 10c.
(18) *Railway and Engineering Review*, Chicago, Ill., 10c.
(19) *Scientific American Supplement*, New York City, 10c.
(20) *Iron Age*, New York City, 10c.
(21) *Railway Engineer*, London, England, 25c.
(22) *Iron and Coal Trades Review*, London, England, 25c.
(23) *Bulletin*, American Iron and Steel Assoc., Philadelphia, Pa.
(24) *American Gas Light Journal*, New York City, 10c.
(25) *American Engineer*, New York City, 20c.
(26) *Electrical Review*, London, England.
(27) *Electrical World*, New York City, 10c.
(28) *Journal*, New England Water-Works Assoc., Boston, Mass., $1.
(29) *Journal*, Royal Society of Arts, London, England, 15c.
(30) *Annales des Travaux Publics de Belgique*, Brussels, Belgium.
(31) *Annales* de l'Assoc. des Ing. Sortis des Ecoles Spéciales de Gand, Brussels, Belgium.
(32) *Mémoires et Compte Rendu des Travaux*, Soc. Ing. Civ. de France, Paris, France.
(33) *Le Génie Civil*, Paris, France.
(34) *Portefeuille Economiques des Machines*, Paris, France.
(35) *Nouvelles Annales de la Construction*, Paris, France.
(37) *Revue de Mécanique*, Paris, France.
(38) *Revue Générale des Chemins de Fer et des Tramways*, Paris, France.
(41) *Modern Machinery*, Chicago, Ill., 10c.
(42) *Proceedings*, Am. Inst. Elec. Engrs., New York City, 50c.
(43) *Annales des Ponts et Chaussées*, Paris, France.
(44) *Journal*, Military Service Institution, Governors Island, New York Harbor, 50c.
(45) *Mines and Minerals*, Scranton, Pa., 20c.
(46) *Scientific American*, New York City, 8c.
(47) *Mechanical Engineer*, Manchester, England.
(48) *Zeitschrift*, Verein Deutscher Ingenieure, Berlin, Germany.
(49) *Zeitschrift für Bauwesen*, Berlin, Germany.
(50) *Stahl und Eisen*, Düsseldorf, Germany.
(51) *Deutsche Bauzeitung*, Berlin, Germany.
(52) *Rigasche Industrie-Zeitung*, Riga, Russia.
(53) *Zeitschrift*, Oesterreichischer Ingenieur und Architekten Verein, Vienna, Austria.
(54) *Transactions*, Am. Soc. C. E., New York City, $5.
(55) *Transactions*, Am. Soc. M. E., New York City, $10.
(56) *Transactions*, Am. Inst. Min. Engrs., New York City, $5.

(57) *Colliery Guardian*, London, England.
(58) *Proceedings*, Engrs.' Soc. W. Pa., 803 Fulton Bldg., Pittsburg, Pa., 50c.
(59) *Transactions*, Mining Inst. of Scotland, London and Newcastle-upon-Tyne, England.
(60) *Municipal Engineering*, Indianapolis, Ind., 25c.
(61) *Proceedings*, Western Railway Club, 225 Dearborn St., Chicago, Ill., 25c.
(62) *Industrial World*, 59 Ninth St., Pittsburg, Pa.
(63) *Minutes of Proceedings*, Inst. C. E., London, England.
(64) *Power*, New York City, 20c.
(65) *Official Proceedings*, New York Railroad Club, Brooklyn, N. Y., 15c.
(66) *Journal of Gas Lighting*, London, England, 15c.
(67) *Cement and Engineering News*, Chicago, Ill., 25c.
(68) *Mining Journal*, London, England.
(70) *Engineering Review*, New York City, 10c.
(71) *Journal*, Iron and Steel Inst., London, England.
(73) *Electrician*, London, England, 18c.
(74) *Transactions*, Inst. of Min. and Metal., London, England.
(75) *Proceedings*, Inst. of Mech. Engrs., London, England.
(76) *Brick*, Chicago, Ill., 10c.
(77) *Journal*, Inst. Elec. Engrs., London, England.
(78) *Beton und Eisen*, Vienna, Austria.
(79) *Forscherarbeiten*, Vienna, Austria.
(80) *Tonindustrie Zeitung*, Berlin, Germany.
(81) *Zeitschrift für Architektur und Ingenieurwesen*, Wiesbaden, Germany.
(82) *Dinglers Polytechnisches Journal*, Berlin, Germany.
(83) *Progressive Age*, New York City, 15c.
(84) *Le Ciment*, Paris, France.
(85) *Proceedings*, Am. Ry. Eng. and M. of W. Assoc., Chicago, Ill.
(86) *Engineering-Contracting*, Chicago, Ill., 10c.
(87) *Roadmaster and Foreman*, Chicago, Ill., 10c.
(88) *Bulletin* of the International Ry. Congress Assoc., Brussels, Belgium.
(89) *Proceedings*, Am. Soc. for Testing Materials, Philadelphia, Pa.
(90) *Transactions*, Inst. of Naval Archts., London, England.
(91) *Transactions*, Soc. Naval Archts. and Marine Engrs., New York City.
(92) *Bulletin*, Soc. d'Encouragement pour l'Industrie Nationale, Paris, France.
(93) *Revue de Métallurgie*, Paris, France, 4 fr. 50.
(94) *The Boiler Maker*, New York City, 10c.
(95) *International Marine Engineering*, New York City, 20c.
(96) *Canadian Engineer*, Toronto, Canada, 15c.
(97) *Turbine*, Berlin, Germany, 1 Mark.
(98) *Journal*, Engrs. Soc. Pa., 219 Market St., Harrisburg, Pa., 30c.

LIST OF ARTICLES

Bridges.

New Bridge of the Kanawha & Michigan Ry. over the Ohio River at Pt. Pleasant, W. Va.* J. A. Stocker. (13) Sept. 9.
Guard Rails and Deck Construction for Railway Bridges.* (13) Sept. 9.
On the Question of the Strengthening the Bridges with a View to Increasing the Speed of Trains.* Pierre A. Zahariade. (88) Oct.
Swing Bridge across the Weaver Navigation.* (12) Oct. 8.
Modjeski's Report on Design and Construction of the Manhattan Bridge over the East River at New York City.* (13) Oct. 14.
Examples of Practice in Turntable Construction with some Data on Costs.* (Abstract from Comm. Report, Amer. Ry. Bridge and Bldg. Assoc.) (86) Oct. 27.
The River Spans of the St. Louis Municipal Bridge.* (14) Oct. 30.
The Assopos Viaduct, Greece.* (13) Nov. 4.
Highway Bridge over the Miami River at Elizabethtown, Ohio; the Longest Simple Truss Span Bridge in Existence.* H. C. Tyrrell. (96) Nov. 5.
Le Viaduc de Wiesen sur la Ligne de Davos à Filisur. (Haute-Engadine.)* (33) Oct. 16.
Belastungsprobe einer Betongelenkbrücke.* (80) Oct. 19.
Die Erbreiterung der Wilhelmsbrücke und die Erbauung einer Müllverbrennungsanstalt in Frankfurt a. M.* Koelle. (Paper read before the Deutsche Betonverein.) (80) Oct. 21.

Electrical.

The Theory and Application of Motor Converters. H. S. Hallo. (77) Sept.
Extending the Limits of Power Transmission, as Applied both to Long-Distance Transmission of Power and to Power Companies' Areas.* A. M. Taylor. (77) Sept.
Electric Traction Vagabond Currents.* J. G. Cunliffe and R. G. Cunliffe. (77) Sept.
Interpole Designs.* W. B. Hird. (77) Sept.

*Illustrated.

Electrical—(Continued).

The German Standards and Regulations for Electrical Machinery. Norman C. Bridge. (77) Sept.
Rules for Continuous-Current Traction Motors, Adopted by the International Congress for Tramways and Light Railways, Milan, 1906. (77) Sept.
On Magnetic Testing of Iron with Alternating Current.* Albert Campbell. (77) Sept.
The Metal Tungsten as "Valve" Electrode. L. H. Walter. (77) Sept.
Some Notes on California High-Tension Transmission Lines. S. L. Foster. (Abstract of paper read before the Amer. Street and Interurban Ry. Eng. Assoc.) (17) Oct. 8.
Report of the Committee on Power Distribution (Amer. Street and Interurban Ry. Eng. Assoc.)* (17) Oct. 8.
Effects of a Third Harmonic in the E. M. F. Wave of a Three-Phase Generator.* F. M. Denton. (27) Oct. 14.
Recent Patents in Wireless Telegraphy and Telephony.* W. H. Eccles. (73) Serial beginning Oct. 15.
Utilization of Energy from Niagara Falls.* (17) Oct. 21.
Electrical Inventions and the Training of the Electrical Engineer. J. A. Fleming. (Lecture before the Univ. of London.) (11) Oct. 22.
A Duddell Oscillograph for Use on a 50 000-Volt Circuit.* (73) Oct. 22.
Telephone Equipment of the Syracuse, Lake Shore & Northern.* (17) Oct. 23.
Standard Clocks Operated by Wireless.* Alfred Gradenwitz. (19) Oct. 23.
The Transmission of Electrical Energy in Southern California.* (27) Oct. 28.
Calculation of the Voltage Regulation of Transmission Lines.* J. E. Dalemont. (27) Oct. 28.
Determination of the Constants of Instrument Transformers.* P. G. Agnew and T. T. Fitch. (27) Oct. 28.
Electricity from Refuse.* (12) Oct. 29.
The Stepney Borough Council's Generating Station at Blyth's Wharf.* (73) Oct. 29; (12) Oct. 29.
The Selection and Installation of Electric Cables in Collieries. W. R. Morton. (73) Oct. 29.
Protection of Electrical Equipment.* P. M. Lincoln. (42) Nov.
Power-House Economics in Baltimore.* Horatio A. Foster. (3) Nov.
The Berlin System of Central Stations.* Frank Koester. (9) Nov.
Electrical Equipment of the Luzerne County Court House.* (27) Nov. 4.
Electrical and Mechanical Equipment of the Blackstone Hotel, Chicago.* (27) Nov. 4.
Etude des Dynamos Commandées par Turbines à Vapeur.* F. Drouin. (33) Oct. 2.

Marine.

Diverging Waves.* William Hovgaard. (90) Vol. 51.
Report of the Experimental Tank Committee, 1908. (Inst. of Naval Archts.)* (90) Vol. 51.
On the Resistance of Thin Plates and Models in a Current of Water. T. E. Stanton. (90) Vol. 51.
The Vibrations of Ships and the Use of a Dynamical Model for Determining the Elasticity of Ships.* J. B. Henderson. (90) Vol. 51.
Some Considerations on the Application of Internal Combustion Engines for Marine Propulsion. H. C. Anstey. (90) Vol. 51.
Internal Combustion Engines for Submarines.* F. R. S. Bircham. (90) Vol. 51.
The Propulsion of Ships by Means of Contrary Turning Screws on a Common Axis.* G. Rota. (90) Vol. 51.
Notes on a Mechanical Method for Determining the Thrust of Propellers.* J. H. Heck. (90) Vol. 51.
Description of the Suction Dredge *Leviathan* recently Constructed for the Port of Liverpool.* Anthony G. Lyster. (90) Vol. 51.
The Turbine Passenger Steamer *Ben-My-Chree*, and Practical Experience of the Parsons Marine Steam Turbine.* C. J. Blackburn. (90) Vol. 51.
The Accelerated Motion of Bodies in Water, with Special Application to the Rolling of Ships.* A. W. Johns. (90) Vol. 51.
On Launching Calculations with Special Reference to the Effect of Camber.* John Smith. (90) Vol. 51.
A Note on Ship Geometry.* George Greenhill. (90) Vol. 51.
Some Points in Connection with Shipbuilding on the Great Lakes, U. S. A.* Herbert C. Sadler. (90) Vol. 51.
The Influence of Form and Bulkheads on the Strength of Ships.* J. Bruhn. (90) Vol. 51.
The Early History of Steam Navigation.* (12) Oct. 8.
German Shipyards.* (12) Serial beginning Oct. 8.
The Italian Battleship *Roma*.* Dagnino Attilio. (95) Nov.
Channel Steamers.* Arthur Barter. (10) Nov.

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Marine—(Continued).

Machinery and Piping Arrangements on Board Ship.* John M'Coll. (95) Serial beginning Nov.
Les Problèmes de Navigation Résolus par la Géométrie Analytique Radiale. Maurice Alliaume. (30) Oct.
Le Service de Ferry-Boats entre Sassnitz (Allemagne) et Trelleborg (Suède).* (33) Oct. 23.

Mechanical.

Explosions of Steam Pipes due to Water Hammer. C. E. Stromeyer. (90) Vol. 51.
Experiments upon the Forces Acting on Twist-Drills When Operating on Cast-Iron and Steel.* Dempster Smith and R. Poliakoff. (75) Mar.
Some Experiments on Lubrication.* W. W. F. Pullen and W. T. Finlay. (75) May.
Machine Tool Design, Modern British Practice.* Fred. Coxon. (98) Aug.
The Electrification of Steam-Driven Non-Reversing Rolling Mills. W. F. Mylan. (77) Sept.
Tests of a High-Efficiency High-Speed Hydraulic Turbine Runner. Ernst Reichel. (13) Sept. 9.
Vitrified Brick; its Constitution and Manufacture. George W. Kummer. (Paper read before the 1st Amer. Cong. of Road Builders.) (76) Oct.
Possibilities of the Commercial Automobile.* W. W. McFarren. (58) Oct.
Steam Turbines.* Gerald Stoney. (29) Serial beginning Oct. 8.
First Impressions of Heavy Charges. G. M. Gill. (66) Oct. 12.
New Works at Wolverhampton.* (12) Oct. 15.
Cylinder Condensation and the Temperature of Cylinder Walls.* R. Royds. (12) Oct. 15.
Retort Benches. A. S. B. Little. (Paper read before the Mich. Gas Assoc.) (83) Oct. 15.
Recent Progress in Gas Manufacture.* Thomas Holgate. (11) Serial beginning Oct. 15.
New Calibration of the Phœnixville Testing Machine. (14) Oct. 16.
Coal and Ore Loading Plant, New Rhine Harbor.* J. B. Van Brussel. (16) Oct. 16.
Recent Practice in Coal and Coke Handling in British Gas Works.* James I. Seager. (24) Serial beginning Oct. 18.
Power House of the Rock Island R. R. Shops.* Osborn Monnett. (64) Oct. 19.
A New Gas Producer for Low Grade Fuel.* Alfred Gradenwitz. (64) Oct. 19.
Sixteen Months' Record at Redondo.* (64) Oct. 19.
An Electrically-Operated Quarry and Plant for Production of Broken Stone at Gary, Ill.* (13) Oct. 21.
Notes on the Design of Shafts, with Special Reference to High-Speed Electric Generators. H. H. Jeffcott. (12) Serial beginning Oct. 22.
Power Plant of the Coronet Phosphate Company.* H. D. Mendenhall. (14) Oct. 23.
A Large Coal Dock at Duluth.* (14) Oct. 23.
The Humphrey Gas-Pump.* (11) Oct. 15; (12) Oct. 22; (47) Oct. 22; (66) Oct. 26.
A Modern Ice Plant in Chicago.* Carl Heim. (64) Oct. 26.
Aerial Flight.* F. W. Lanchester. (29) Serial beginning Oct. 29.
Exhaust Steam Turbines in Collieries and Rolling Mills.* A. P. Chalkley. (73) Oct. 29.
Low-Tension Ignition-Gear for Gas-Engines.* (11) Oct. 29.
Electrically-Driven Reversing Rolling Mills.* C. A. Ablett. (Paper read before the Inst. of Engrs. and Shipbuilders in Scotland.) (47) Serial beginning Oct. 29.
Mechanical Equipment of the Washington Municipal Building.* (14) Oct. 30.
A Service Test of Internally Fired Boilers. (14) Oct. 30.
The Design of Turning Engines. Edward M. Bragg. (95) Nov.
Efficiency in the Purchase of Fuel for Power Generation.* Walter N. Polakov. (9) Nov.
The Mitchell Patent Coke Oven.* John Fulton. (45) Nov.
Test of Staybolt Threads.* C. J. Morrison. (25) Nov.
The Loss of Heat from Uncovered Steam Pipes.* William Ledyard Cathcart. (10) Nov.
Italian Boiler Requirements. (From *Consular and Trade Reports.*) (94) Nov.
A New and Efficient Jet Condenser.* (13) Nov. 4.
The Agreement of Theory and Practice in Steam Boiler Tests.* Frank Kingsley. (14) Nov. 6.
Recherches sur le Fonctionnement des Injecteurs.* G. Schrauff. (37) Sept. 30.
Des Vérifications à Faire Subir aux Machines-Outils. A. Leboulle. (30) Oct.
Sur la Cémentation par le Carbone.* Léon Guillet et Charles Griffith. (93) Oct.

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Contribution à l'Etude des Aciers pour Engrenages. Louis Révillon. (93) Oct.
La Turbine à Vapeur Oerlikon.* Ch. Dantin. (33) Oct. 9.
Premier Congrès Français du Froid (Lyon, 1er-3 Octobre, 1909). E. Lemaire. (33) Serial beginning Oct. 16.
L'Exposition Internationale de Locomotion Aérienne (Paris, 25 Septembre-17 Octobre, 1909).* G. Espitallier. (33) Oct. 23.
Die Internationale Luftschiffahrt-Ausstellung in Frankfurt a. M.* A. Varreiter. (48) Serial beginning Oct. 2.
Versuche an der Kohlenumladeanlage in Kosel.* Kammerer. (48) Serial beginning Oct. 2.
Eine Gottländische Portlandzementfabrik.* H. F. Boye. (80) Oct. 2.
Ueber Dampfturbinen-Kondensation.* Wilhelm Geratsch. (97) Serial beginning Oct. 5.
Ueber die Verwendung von Braunkohlenbriketts in Eisen- und Stahlgiessereien.* K. Krumbiegel. (Paper read before the Deutsche Giessereifachleute.) (50) Oct. 6.
Qualitätsforderungen für Giessereikoks. Oscar Simmersbach. (Paper read before the Deutscher Giessereifachleute.) (50) Oct. 6.
Neuere Ausführungen von Kompressoren.* E. W. Köster. (48) Serial beginning Oct. 9.
Drehrohrofen und Zerkleinerungsapparate.* (80) Oct. 12.
Die Bedeutung von Modellversuchen für die Luftschiffahrt und Flugtechnik und die Einrichtung für Solche Versuche in Göttingen.* L. Prandtl. (48) Oct. 16.
Die Buderus'schen Eisenwerke zu Wetzlar.* (50) Oct. 20.

Metallurgical.

Relation of Electric Furnaces to Siderurgy.* P. McN. Bennie. (98) Aug.
The Girod Electric Furnace for the Manufacture of Steel.* Paul Girod. (Abstract of paper read before the Amer. Electrochemical Soc.) (73) Oct. 8.
The Kjellin and Röchling-Rodenhauser Electric Furnaces.* F. A. Kjellin. (Abstract of paper read before the Amer. Electrochemical Soc.) (73) Oct. 8.
A Comparison of the Advantages of Smelting Iron Ore in the Electric Furnace and in the Blast Furnace. J. Hardén. (Abstract from *Electrochemical and Metal. Ind.*) (73) Oct. 22.
Iron and Steel; Their Relation to other Industries. F. W. Harbord. (Paper read before the Soc. of Chem. Ind.) (19) Serial beginning Oct. 23.
Concentration of Flake Graphite. Frederick D. Chester. (16) Oct. 23.
Copper-Clad Steel. Wirt Tassin. (From *Jour. Ind. and Eng. Chem.*) (16) Oct. 23.
Modern Type of the Barrel Copper Converter.* Charles F. Shelby. (16) Oct. 23.
Handling Stamp Mill Tailings by Belt Conveyors.* (13) Oct. 28.
Ore Dressing in the United States and Mexico.* H. A. Guess. (16) Serial beginning Oct. 30.
Wastes of Heat and Materials in Smelting Works. Herbert Lang. (16) Nov. 6.

Military.

Graphical Illustrations of Professor Kaiser's Theory on Calculating Stresses and Strains in Built-Up Guns.* A. Bremberg. (11) Serial beginning Oct. 29.

Mining.

Mine Gases and Colliery Explosions. H. B. Winstanley. (Paper read before the National Assoc. of Colliery Mgrs.) (22) Oct. 8.
Method and Cost of Sinking a Mining Shaft through Quick Sand using a Steel Shoe. (86) Oct. 13.
Electric Winding at the Ferndale Collieries.* (22) Oct. 15.
New Type of Hoisting Engine.* (16) Oct. 16.
Electrification of the Ferndale Collieries.* (22) Oct. 22; (73) Oct. 29.
The Klonne Type of High-Level Storage Tank.* A. Gradenwitz. (16) Oct. 23.
Notes on the Effect of Earthquakes on Deep Underground Water Circulation. W. H. Yeandle. (16) Oct. 30.
Estimated Cost of Sinking Shaft. (45) Nov.
Applications of Concrete in the Metal-Mining Industry.* Henry W. Edwards. (9) Nov.
Marquette-Range Caving Method.* H. H. Stoek. (45) Nov.
Upper Potomac Coal Fields. H. H. Stoek. (45) Nov.
A New Ventilating Fan for Mines.* M. C. Mitchell. (Abstract from *Mine and Quarry*). (45) Nov.
Transportation of Coal by Flume.* Robert M. Magraw. (45) Nov.
Expériences sur le Rendement et le Mode d'Emploi des Marteaux à Air Comprimé dans les Mines. (Abstract from *Bul.* de la Soc. de l'Ind. Min.) (33) Oct. 9.

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Miscellaneous.

The Measurement of Humidity. Samuel K. Patteson. (10) Nov.
The Utilization of Atmospheric Nitrogen. A. Bernthsen. (Abstract of paper read before the 7th Inter. Cong. of Applied Chemistry.) (16) Oct. 16.
Low Cost of Excavation with Fresno Scrapers.* Walter N. Frickstad. (86) Nov. 3.

Municipal.

Cost of Pavements. (Report to Ill. Soc. of Engrs. and Survs.) (76) Oct.
Asphalt Pavements (Cost). (96) Oct. 15.
Street Grades.* (14) Oct. 16.
Brick Pavement for the Indianapolis Motor Speedway.* (14) Oct. 16.
An Electrically-Operated Quarry and Plant for Production of Broken Stone at Gary, Ill.* (13) Oct. 21.
Portable Machines for Making Asphaltic Concrete Paving.* (13) Oct. 21.
Dust Prevention by Calcium Chloride. (96) Oct. 22.
Method of Constructing Creosoted Wood Block Pavement at Grand Forks, N. D. (Abstract of paper read before the Ill. Soc. of Engrs. and Survs.) (86) Oct. 27.
Tar in the Surface Treatment of Roads. Harvey J. Skinner. (Paper read before the Amer. Gas Inst.) (60) Nov.; (96) Oct. 22.
Methods and Some Costs on the Construction of Chert-Macadam Roads. Edward B. Kay. (Abstract of paper read before the Ala. Good Roads Assoc.) (86) Nov. 3.
The Road Surface. W. A. McLean. (Paper read before the Ontario Land Survs.) (96) Nov. 5.

Railroads.

Address by the President (Electrification of Railways.)* John A. F. Aspinall. (75) Apr.
The Luminous Repeating Signals for Fog on the Brussels-Antwerp Line.* L. Weissenbruch and J. Verdeyen. (88) Oct.
On the Question of Improvements in Locomotive Boilers.* H. Fowler and L. Archbutt. (88) Oct.
On the Question of the Working of Light Railways. Plocq. (88) Oct.
Automatic Sanding Apparatus for the Westinghouse Brake.* S. Brütsch. (88) Oct.
The Moving of Signal Cabin "A" at Utrecht Central Station.* A. A. Boon. (From *De Ingenieur.*) (88) Oct.
Superheated Steam Locomotive, San Paulo (Brazilian) Railway. (12) Oct. 8; (15) Oct. 29.
Some Recent Locomotive Performances on the Northern Railway of France. J. T. Burton-Alexander. (12) Oct. 8.
Traction System Using Constant Direct Current.* J. E. Dalemont. (27) Oct. 14.
The Works of Kerr, Stuart and Co., Limited, at Stoke-on-Trent.* (12) Oct. 15.
Locomotive Weighing-Machine with Automatic Indicating Quadrants.* (11) Oct. 15.
Electric Locomotives for the British Columbia Electric Railway.* (96) Oct. 15.
Walschaerts Valve Gear on Lackawanna Consolidation Locomotive.* (15) Oct. 15.
Erie Railroad Terminal Improvements at Jersey City.* (14) Oct. 16.
The Beebee Syndicate Interurban System in Central New York; Power Supply, Equipment and Traffic Features.* (17) Oct. 16.
The Scranton Locomotive Shops of the Lackawanna Railroad.* (14) Oct. 16.
Electric Lighting on European Railroads. Max Buttner. (Abstract of paper read before the Assoc. of Car Lighting Engrs.) (18) Oct. 23.
Working Conductors for Electric Railways.* (Abstract of report to Amer. St. and Interurban Ry. Eng. Assoc.) (18) Oct. 16.
The Signalling Installation at Yarnton on the Great Western Railway.* (73) Oct. 22.
The Ashendon-Aynho Railway.* (12) Oct. 22.
Telephone Equipment of the Syracuse, Lake Shore & Northern.* (17) Oct. 23.
An Armored Concrete Viaduct, Rotterdam's Remarkable Grade Crossings.* C. Van Langendouck. (19) Oct. 23.
The Madeira-Mamoré Railway.* John Y. Bayliss, M. Am. Soc. C. E. (13) Oct. 28.
The Southern Pacific Taper Curve Tables and their Use with the Traverse Method.* Wm. Clyde Willard. (13) Oct. 28.
Rail Grooves and Track Gage for Curved Tracks.* C. W. L. Filkins. (Report of the Comm. on Way Matters.) (96) Oct. 29.
Test of the Jacobs Superheater on the Santa Fe.* (15) Oct. 29.
Reconstruction and Improvements of the Los Angeles Pacific Company.* P. H. Albright. (17) Oct. 30.
Grade Crossing Elimination in Worcester.* C. H. Gilman. (14) Oct. 30.

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The Electric System of the Great Northern Railway Company at Cascade Tunnel.* Cary T. Hutchinson. (42) Nov.
Latter-Day Developments of the American Locomotive.* H. Keith Trask. (9) Nov.
Double Open Diagonal Truss for Steel Passenger Cars, on Four Points of Support, and with Equal Overhanging Ends; Analytical and Graphical Analysis.* Arthur E. Heffelfinger. (25) Nov.
Pennsylvania Electric Locomotives.* (15) Nov. 5; (27) Nov. 4; (17) Nov. 6; (18) Oct. 30.
Scranton Shops of the Delaware, Lackawanna & Western.* George L. Fowler. (15) Nov. 5.
Mallet Articulated Passenger Locomotives for the A., T. & S. F. Ry.* (18) Nov. 6.
Railroad Coaling Stations and Cinder Pits. (Abstract of report made to the Amer. Ry. Bridge and Bldg. Assoc.) (14) Nov. 6.
New Boston Freight Terminal, Boston and Albany R. R.* (14) Nov. 6.
New Car Repair Shops of the Oneida Railway Company at Syracuse, N. Y.* (17) Nov. 6.
The Loetschberg Tunnel Through the Bernese Alps.* Alfred Gradenwitz. (19) Nov. 6.
Role, Répartition et Calcul des Actions Horizontales dans les Voies des Chemins de Fer.* L. Schlussel. (32) Sept.
La Voie Courante des Chemins de Fer de l'Etat Belge.* Pierre Decamps. (38) Oct.
Note sur les Chemins de Fer de l'Afrique Occidentale Française.* R. Godfernaux. (38) Oct.
Bogies Extensibles pour Matériel Roulant à Voie Normale et à Voie de 1 Métre.* (33) Oct. 2.
Types Récents de Foyers pour Chaudières de Locomotives.* M. Emanaud. (33) Oct. 2.
Der Einfluss des Titans auf Stahl, besonders auf Schienenstahl.* Ed. von Maltitz. (50) Oct. 13.

Railroads, Street.

The Electrical System of the London County Council Tramways.* John Hall Rider. (77) Sept.
Phases in the Development of the Street Railways of Chicago.* Bion J. Arnold, H. B. Fleming and George Weston. (4) Oct.
Reconstruction of Street Car Tracks in Chicago.* George Weston. (4) Oct.
Tests of Line Insulators.* (Abstract of report to Amer. Street and Interurban Ry. Eng. Assoc.) (17) Oct. 9.
Recent Improvements on the Hamburg-Blankensee-Ohlsdorf High-Tension, Single-Phase Railway.* (17) Oct. 23.
Coasting Registers on the Manhattan Elevated.* (17) Oct. 30.
Fiber Brakeshoe used on the London Underground Railways.* (17) Oct. 30.
The Arborway Section of the Boston Elevated Railway.* (14) Nov. 6.
The Street Railway System of Bombay, India.* (17) Nov. 6.
Roadbed and Track Repairs in Detroit.* (17) Nov. 6.

Sanitation.

Disinfection of Sewage with Chloride of Lime.* F. E. Daniels. (98) Aug.
A Comparison of Contact Beds and Percolating Filters.* John E. Farmer. (From *The Surveyor.*) (13) Sept. 9.
Hammer Drills Applied to Rock Excavation in Sewer Construction at Bloomington, Ind.* Geo. L. Thon. (13) Sept. 9.
Heating and Ventilation of the Technical High School, Newton, Mass.* Charles L. Hubbard. (70) Oct.
Advantages of Large Size Drain Tile. Frank M. Okey. (Paper read before the Iowa Brick and Tile Assoc.) (76) Oct.
Methods and Costs of Drainage Work in Illinois.* (Abstract from report to Ill. Soc. of Engrs. and Surv.) (86) Oct. 13.
Some Notes on the Louisville Sewerage Improvements.* Roger DeL. French. (13) Oct. 14.
The Cesspool and its Dangers. Burton J. Ashley. (Paper read before the Amer. Soc. of Inspectors of Plumbing and San. Engrs.) (96) Oct. 15.
The Washington Street Sewer at Lynn.* (14) Oct. 16.
Heating and Ventilation in the Ford Building, Detroit.* (14) Oct. 16.
Costs of Catch Basin Construction by Contract and by Day Labor in Boston.* (Abstract from report Boston Finance Comm.) (86) Oct. 20.
Reports by Col. W. M. Black on the Pollution of New York Harbor and the Proposed Passaic Valley and Bronx Valley Sewer Outlets. (13) Oct. 21; (14) Oct. 23.
Sizes, Materials and Shapes of Sewers in the City of Hamilton.* E. G. Barrow. (Paper read before the Ontario Land Surveyors.) (96) Oct. 29.
Electricity from Refuse.* (12) Oct. 29.
The Sewage Pumping Plant at Baltimore. (14) Oct. 30.

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Sewer Construction in Stapleton, Staten Island.* (14) Oct. 30.
Some Structural Details and Notes on Construction of a Reinforced Concrete Pipe Sewer.* (86) Nov. 3.
Molded Concrete Pipe and Storm Drains, Newark, New Jersey.* (14) Nov. 6.
A Concrete and Steel Storm Water Sewer and Some Remarks on Specifications and Designs.* (86) Nov. 3.
Theory and Practice in the Use of the Dibdin Slate Beds for Sewage Disposal.* W. J. Dibdin. (Abstract of paper read before the Inst. of San. Engrs.) (86) Nov. 3; (14) Nov. 6.
Die Arbeiten der Emschergenossenschaft.* Middeldorf. (51) Serial beginning Sept. 29.
Vergleich der Ausdehnungsfähigkeit einer Warmwasser Fernheizung gegenüber einer Hochdruckdampf-Fernheizung. Schröder. (51) Oct. 9.

Structural.

Work of Completing Chalmette Monument.* Alfred F. Theard. (Paper read before the Louisiana Eng. Soc.) (1) Sept.
Costs of Constructing Combination Concrete and Tile Floors in Three Buildings. W. K. Hatt. (Abstract of paper read before the Indiana Eng. Soc.) (86) Oct. 13.
Method and Cost of Sheeting a Foundation Pit with Steel Sheet Piling. (86) Oct. 13.
A Comparison of the Various Processes of Preserving Timber.* G. B. Shipley. (13) Oct. 14.
Foundation Waterproofing in the State Education Building.* (14) Oct. 16.
The Nature of Portland Cement. Edward Godfrey. (14) Oct. 16.
Reinforced Concrete for Factory Construction.* J. P. H. Perry. (20) Oct. 21.
Copper-Clad Steel. Wirt Tassin. (From *Jour. Ind. and Eng. Chem.*) (16) Oct. 23.
Unusual Cellar Waterproofing.* (14) Oct. 23.
A Concrete Church and the Methods of Waterproofing it.* (14) Oct. 23.
Methods of Executing Light Work at Great Heights.* (14) Oct. 23.
Paints for Metal with Comments on the Use of Compressed Air for Painting and Cleaning. Geo. L. Fowler, M. Am. Soc. C. E. (Report to Amer. Bridge and Bldg. Assoc.) (86) Oct. 27.
Observations on Materials for Binding Mortar. Anton Hambloch. (12) Oct. 29.
The Elastic Breakdown of Non-Ferrous Metals.* C. Alfred M. Smith. (Paper read before the Inst. of Metals.) (11) Serial beginning Oct. 29; (47) Serial beginning Oct. 29.
The Compressive Strength of Coke Concrete.* John M. Lewis. (14) Oct. 30.
The Rate of Increase in Strength of Cement. (Report by Clifford Richardson.) (14) Oct. 30.
Concreting Structural Foundations. F. A. Lart. (10) Nov.
Calculations for Determining the Size of Plates for a Self-Supporting Steel Stack Base.* J. N. Heltzel. (94) Nov.
Improvements in Stress Indicators.* (96) Nov. 5.
Erecting a Large Dome without Falsework.* (14) Nov. 6.
Pneumatic Foundations of the Emigrant Bank Building.* (14) Nov. 6.
Dalles Creuses en Ciment Armé. (84) Sept.
Accidents aux Constructions en Béton Armé. Fr. v. Emperger. (Report to Inter. Assoc. for Testing Materials, 5th Cong.) (84) Sept.
Récentes Innovations dans la Mouture des Ciments. (84) Sept.
Laiton à Cartouches, Laiton à Balles, Cuivre Electrolytique. Grard. (93) Oct.
Allure des Courbes Réliant la Dureté d'un Acier à la Température de Recuit après Trempe ou après Ecrouissage.* Grenet. (93) Oct.
Le Procédé Elmore d'Enrichissement des Minerais par le Vide et ses Résultats Industriels.* L. Descroix. (93) Oct.
Poutre à Treillis en Béton Armé système Vistini.* Bousquet. (35) Oct.
Hangar Métallique Démontable, Système Vaniman pour Ballons Dirigeables.* A. Bidault des Chaumes. (33) Oct. 2.
Le Passage Couvert de la Friedrichstrasse, à Berlin.* (33) Oct. 9.
Zur Chemischen Analyse des Portlandzementes. (80) Oct. 2.
Ueber Versuche mit Vorsatzbeton.* Carl Meyer. (78) Oct. 8.
Beitrag zur Beurteilung der Wirkung Ruhender Lasten auf Rostpfähle.* E. v. Willmann. (78) Oct. 8.
Die Eisenbetonkonstruktionen im Johann-Strauss-Theater in Wien.* Richard Hoffmann. (78) Oct. 8.
Die Wirkung des Temperaturwechsels auf Mauerwerk.* (78) Serial beginning Oct. 8.
Die Bewertung der Höhe des Kalkgehaltes im Zement. H. Trachler. (Paper read before the Verein Deutscher Portlandzementfabrikanten.) (80) Oct. 14.

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Topographical.

A Description of Four Stadia Surveys and their Cost.* Arthur W. Tidd, Assoc. M. Am. Soc. C. E. (13) Oct. 21.

Water Supply.

Water Filtration and Sterilization. George A. Johnson. (98) Aug.
"Sterilization" of a Potable Water Supply by Means of "Bleach." J. L. Leal. (98) Aug.
A Reinforced-Concrete Spillway with Concentrated Crest Length.* D. W. Murphy. (13) Sept. 9.
Some Observations on the Stability of Dams.* J. F. Jackson. (4) Oct.
The Failure of the Fergus Falls, Minn., City Dam.* Franklin McMillan. (13) Oct. 14.
The Cost of Clearing Water in Settling Basins. S. Bent Russell. M. Am. Soc. C. E. (Abstract of paper read before the Central States Water Works Assoc.) (13) Oct. 14; (14) Oct. 16; (86) Oct. 20.
Guelph's Water Supply.* A. E. Oakley. (96) Oct. 15.
Water Filtration Plant at Sandusky, Ohio.* (14) Oct. 16.
The Standley Lake Irrigation System near Denver, Colo.* (14) Serial beginning Oct. 16.
A New World's Record of Progress in Rock Tunneling; Red Rock Tunnel, Los Angeles Aqueduct.* (86) Oct. 20.
Experimental Data on Methods and Costs of Draining Irrigated Lands.* (Abstract of Farmer's Bulletin. U. S. Dept. of Agri.) (86) Oct. 20.
Water Waste Prevention by Adequate Surveys and Inspection. (86) Oct. 27.
Philadelphia's High Pressure Fire System Extended.* (20) Oct. 28.
Hydroelectric Development of the Uncas Power Company at Scotland, Conn.* (27) Oct. 28.
Low-Head Hydroelectric Development on Tippecanoe River at Monticello, Ind.* (17) Oct. 30.
Tumwater Power Plant of the Great Northern Railway.* (14) Oct. 30.
The Interpretation of Chemical Water Analysis. Hartwig Klut. (Tr. by E. A Gieseler.) (14) Oct. 30.
Hydraulic Turbine Supply.* C. W. Harris. (Abstract of paper read before the Pacific Northwest Soc. of Engrs.) (18) Oct. 30.
A Combined Water-Supply, Irrigation, and Power Project.* Burt A. Heinly. (9) Nov.
The Los Angeles Aqueduct.* Burt A. Heinly. (60) Nov.
The Interference of Wells. Frederick G. Clapp. (13) Nov. 4.
Ozone Treatment; Report by Toronto City Engineer to the Corporation. (96) Nov. 5.
The New Filtration Plant for Wilkinsburg. F. B. Leopold. (Paper read before the Central States W. W. Assoc.) (96) Nov. 5.
Underground Water in Crystalline Rocks.* Frederick G. Clapp. (14) Nov. 6.
A Twelve-Mile Concrete Pipe Conduit.* (14) Nov. 6.
Conservation of Plane Section in Concrete Beams.* Roland P. Davis. (14) Nov. 6.
Irrigation de la Mésopotamie, Projets de Sir William Willcocks.* Louis Godard. (33) Oct. 16.
Der Betrieb einer Wasserkraftanlage im Winter. W. v. Winkler. (From *Elektrotechnik und Maschinenbau.*) (97) Serial beginning Oct. 5.
Wirkungsgrad Hydraulischer Akkumulierungspumpanlagen.* A. Pfarr. (53) Oct. 22.

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Concrete Pier Construction on the Pacific Coast; the New Reinforced-Concrete Wharf at Santa Monica. H. H. Dunn. (10) Nov.
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*Illustrated.

PROCEEDINGS

OF THE

AMERICAN SOCIETY

OF

CIVIL ENGINEERS

(INSTITUTED 1852)

VOL. XXXV—No. 10

DECEMBER, 1909

Edited by the Secretary, under the direction of the Committee on Publications.

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NEW YORK 1909

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American Society of Civil Engineers

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ON ENGINEERING EDUCATION: Desmond FitzGerald, Benjamin M. Harrod, Onward Bates, D. W. Mead, Charles Hansel.

ON STEEL COLUMNS AND STRUTS: Austin L. Bowman, Alfred P. Boller, Emil Gerber, Charles F. Loweth, Ralph Modjeski, Frank C. Osborn, George H. Pegram, Lewis D. Rights, George F. Swain, Emil Swensson, Joseph R. Worcester.

ON BITUMINOUS MATERIALS FOR ROAD CONSTRUCTION: W. W. Crosby, A. W. Dean, H. K. Bishop, A. H. Blanchard.

The House of the Society is open from 9 A.M. to 10 P.M. every day, except Sundays, Fourth of July, Thanksgiving Day, and Christmas Day.

HOUSE OF THE SOCIETY—220 WEST FIFTY-SEVENTH STREET, NEW YORK.

TELEPHONE NUMBER.....5913 Columbus.
CABLE ADDRESS....."Ceas, New York."

Vol. XXXV. DECEMBER, 1909. No. 10.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PROCEEDINGS

This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

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MINUTES OF MEETINGS

OF THE SOCIETY

November 17th, 1909.—The meeting was called to order at 8.30 P. M.; Vice-President George H. Pegram in the chair; Chas. Warren Hunt, Secretary; and present also, 140 members and 24 guests.

A paper by Julio F. Sorzano, M. Am. Soc. C. E., entitled "Water Supply for the Lock Canal at Panama" was presented by the Secretary. The paper was discussed orally by C. E. Grunsky, M. Am. Soc. C. E., and communications on the subject from Messrs. H. F. Hodges and Theodore Paschke, were presented by the Secretary.

A paper by F. E. Trask, M. Am. Soc. C. E., entitled "The Outlet Control of Little Bear Valley Reservoir," was presented by title.

The Secretary announced the death of ALBERT STANLEY RIFFLE, elected Member May 7th, 1890; died October 28th, 1909.

Adjourned.

December 1st, 1909.—The meeting was called to order at 8.30 P. M.; Vice-President George H. Pegram in the chair; Chas. Warren Hunt, Secretary; and present, also, 212 members and 42 guests.

A paper by James H. Brace and Francis Mason, Members, Am. Soc. C. E., entitled "The New York Tunnel Extension of the Pennsylvania Railroad: The Cross-Town Tunnels," was presented by Mr. Mason and illustrated with lantern slides.

The paper was discussed by Edward Wegmann, M. Am. Soc. C. E.

The Secretary announced the election of the following candidates by the Board of Direction on November 30th, 1909:

AS HONORARY MEMBER.

HIRAM FRANCIS MILLS, Lowell, Mass.

AS MEMBERS.

LYNN LEROY DAVIS, Buffalo, N. Y.
ARCHIBALD STEWART DOWNEY, Seattle, Wash.
HARRY MADERA GOULD, Nashville, Tenn.
LOUIS JENISON HOTCHKISS, Chicago, Ill.
TIEN YOW JEME, Peking, China.
HUDSON FLACK LAYTON, Pittsburg, Pa.
WILLIAM AUGUSTUS MCCALLA, Decatur, Ala.
CHARLES TRINDER MIDDLEBROOK, Albany, N. Y.
ABRAHAM MOREAU REYNOLDS, Newark, N. J.
CHARLES ALFRED SHEPPARD, Edwardsville, Ill.
FRANCIS PITT SMITH, New York City.
WILLIAM DOWLIN TYLER, Dante, Va.
EDWARD VAN WINKLE, New York City.
FRANCIS POTTS WITMER, East Orange, N. J.

AS ASSOCIATE MEMBERS.

LOURENCO BAETA-NEVES, Bello Horizonte, Brazil.
GEERT BLAAUW, New York City.
GEORGE GLOVER BLACKMORE, New York City.
IRVING CLINTON BROWER, Nevada, Mo.
JAMES WATSON DAVENPORT, Memphis, Tenn.
BENJAMIN HERMAN DAVIS, Hoboken, N. J.
JAMES MOORE FARRIN, Camaguey Cuba.
ANDREW JACKSON FISK, Jr., St. Ignatius, Mont.
RUSSELL JOHN HAWN, Fordwick, Va.

John Edward Jennings, Milliken, N. Y.
Thomas Edward Jewett, Houston, Tex.
Walter Alpheus Jones, Titusville, Pa.
Reuben Sylvester Peotter, Mercedes, Tex.
Walter Edward Sanford, Elmhurst, N. Y.
Robert Warren Shelmire, Cleveland, Ohio.
Jay Varnum Wescott, Chicago, Ill.

As Associates.

James Vance Howe, Morgantown, W. Va.
Holger Struckmann, St. Louis, Mo.

As Juniors.

Harold Affleck Brainerd, Elmira, N. Y.
Walter Day Cater, Yorktown, Va.
Harold Bowen Edmundson, Thornton, R. I.
Ralph William Hazen, Manila, Philippine Islands.
Walter William Huff, Cleveland, Ohio.
Alfred Kessinger McCampbell, Billings, Mont.
William John Millard, Guatemala City, Guatemala.
Hugh Miller, Potsdam, N. Y.
Edgar Alva Norwood, Rockport, Mass.
Clifford French Phillips, St. Louis, Mo.
Hunter Imboden Snyder, Nashville, Tenn.
Edward Wording Stearns, Wilkinsburg, Pa.
Charles Sumner Stewart, Morris, Minn.
Jay William Toms, New York City.

The Secretary announced the transfer of the following candidates by the Board of Direction on November 30th, 1909:

From Associate Member to Member.

Norman Freed Brown, Pittsburg, Pa.
Edwin Clifford Finley, St. Louis, Mo.
Henry Neely Ogden, Ithaca, N. Y.
DeWitt Lee Reaburn, Surrey, Cal.
John Martin Schreiber, Newark, N. J.
Wilford Ashford Thompson, East St. Louis, Ill.

From Junior to Associate Member.

Wilbur Howard Fisher, Twin Falls, Idaho.
Camden Page Fortney, Gatun, Canal Zone, Panama.

The Secretary announced the death of James Gardner Sanderson, elected Member, July 6th, 1881; died November 15th, 1909.

Adjourned.

December 15th, 1909.—Because of the necessity of going to press with this number of *Proceedings* in advance of this meeting, the publication of its minutes must be deferred until January, 1910. Two papers have been set down for discussion: "The New York Tunnel Extension of the Pennsylvania Railroad: The East River Tunnels," by James Brace, Francis Mason, and S. H. Woodard, Members, Am. Soc. C. E., and "The New York Tunnel Extension of the Pennsylvania Railroad: Contractors' Plant for East River Tunnels," by Henry Japp, M. Am. Soc. C. E.

OF THE BOARD OF DIRECTION

(Abstract)

November 30th, 1909.—Vice-President Swain in the chair; Chas. Warren Hunt, Secretary; and present, also, Messrs. Andrews, Churchill, Endicott, Hazen, Hodgdon, Knap, Schneider, Swensson, Thompson, Tillson, and Williams.

The subscription price to *Proceedings* was fixed at $8 per annum for the future, to begin January 1st, 1910.

A special subscription rate to students of any technical school, for *Proceedings,* was fixed at $4.50 per annum.

Hiram Francis Mills was elected an Honorary Member.

Ballots for membership were canvassed, resulting in the election of 1 Honorary Member, 14 Members, 16 Associate Members, 2 Associates, and 14 Juniors, and the transfer of 2 Juniors to the grade of Associate Member.

Six Associate Members were transferred to the grade of Member.

Applications were considered and other routine business transacted.

Adjourned.

ANNOUNCEMENTS

The House of the Society is open from 9 A. M. to 10 P. M., every day, except Sundays, Fourth of July, Thanksgiving Day, and Christmas Day.

MEETINGS

January 5th, 1910.—8.30 P. M.—At this meeting a paper by Horace J. Howe, M. Am. Soc. C. E., entitled "Notes on the Replacing of the Superstructure of the Harlem Ship Canal Bridge," will be presented for discussion.

This paper was printed in *Proceedings* for November, 1909.

Wednesday and Thursday, January 19th and 20th, 1910.—The Fifty-seventh Annual Meeting will be held. The Business Meeting will be called to order at 10 o'clock on Wednesday morning at the Society House. The Annual Reports will be presented, officers for the ensuing year elected, members of the Nominating Committee appointed, Reports of Special Committees presented for discussion, and other business transacted.

Arrangements for the Annual Meeting have been placed in the hands of a committee composed of Messrs. T. Kennard Thomson, Thaddeus Merriman, and Charles Warren Hunt.

February 2d, 1910.—8.30 P. M.—At this meeting two papers will be presented for discussion, as follows: "Underpinning the Cambridge Building, New York City," by T. Kennard Thomson, M. Am. Soc. C. E., and "Building Agreements," by William B. Bamford, Assoc. M. Am. Soc. C. E.

These papers are printed in this number of *Proceedings*.

February 16th, 1910.—8.30 P. M.—A paper entitled "The Effect of Alkali on Concrete," by George Gray Anderson, M. Am. Soc. C. E., will be presented for discussion.

This paper is printed in this number of *Proceedings*.

SUBSCRIPTION PRICE TO THE PUBLICATIONS OF THE SOCIETY

The following subscription rates have been fixed by the Board of Direction for the publications of the Society:

Proceedings, ten Numbers per annum, $8.

Transactions, four Volumes per annum, $12. Price for single volumes, $4.

On the above prices there is a discount of 25% to members who desire extra copies of any of these publications, to Libraries, and to Book-dealers.

A special subscription rate has been fixed by the Board for the *Proceedings* of the Society for the benefit of Students in Technical

Schools. This rate is $4.50 per annum, and is available to any *bona fide* student of any technical school.

MEETINGS OF THE SAN FRANCISCO ASSOCIATION OF MEMBERS, AM. SOC. C. E.

The San Francisco Association of Members of the American Society of Civil Engineers holds regular bi-monthly meetings, with banquet and weekly informal luncheons. The former are held at 6 P. M., at the Fairmont Hotel, on the third Friday of February, April, June, August, October, and November, and also on the third Wednesday of December, the latter being the Annual Meeting of the Association.

Informal luncheons are held at 12.15 P. M. every Wednesday, and the place of meeting may be ascertained by communicating with the Secretary of the Association, E. T. Thurston, Jr., Assoc. M. Am. Soc. C. E., 623 Wells Fargo Building.

The by-laws of the Association provide for the extension of hospitality to any members of the Society who may be temporarily in San Francisco, and any such member will be gladly welcomed as a guest of the Association at any of the above meetings, if he will notify the Secretary that he is in San Francisco.

PAPERS AND DISCUSSIONS

The first three Volumes of *Transactions* for 1909 (Volumes LXII, LXIII, and LXIV) have been issued. The fourth volume will be issued on December 31st.

It is hoped that members and others who take part in the discussion of the papers presented will revise their remarks promptly, and that all written communications from those who cannot attend the meetings will be sent in at the earliest possible date after the issue of a paper in *Proceedings*. The issue of volumes of *Transactions* is dependent on the closing of discussions, and the co-operation of the membership will now be more necessary in this matter than heretofore, because four volumes are issued during the year, instead of two, and, to accomplish this, a definite date of issue for each has to be established.

All papers accepted by the Publication Committee are classified by the Committee with respect to their availablity for discussion at meetings.

Papers, which, from their general nature, appear to be of a character suitable for oral discussion, will be published as heretofore in *Proceedings*, and set down for presentation to a future meeting of the Society, and, on these, oral discussion, as well as written communications, will be solicited.

All papers which do not come under this heading, that is to say, those which, from their mathematical or technical nature, in the

opinion of the Committee, are not adapted to oral discussion, will not be scheduled for presentation to any meeting. Such papers will be published in *Proceedings* in the same manner as those which are to be presented at meetings, but written discussions, only, will be requested for subsequent publication in *Proceedings* and with the paper in the volumes of *Transactions.*

PRIVILEGES OF ENGINEERING SOCIETIES EXTENDED TO MEMBERS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

Members of the American Society of Civil Engineers will be welcomed by the following Engineering Societies, both to the use of their Reading Rooms and at all Meetings:

American Institute of Mining Engineers, 29 West Thirty-ninth Street, New York City.

Associação dos Engenheiros Civis Portuguezes, Lisbon, Portugal.

Australasian Institute of Mining Engineers, Melbourne, Victoria, Australia.

Boston Society of Civil Engineers, 715 Tremont Temple, Boston, Mass.

Brooklyn Engineers' Club, 117 Remsen Street, Brooklyn, N. Y.

Canadian Society of Civil Engineers, 413 Dorchester Street, West, Montreal, Que., Canada.

Civil Engineers' Society of St. Paul, St. Paul, Minn.

Cleveland Engineering Society, 718 Caxton Building, Cleveland, Ohio.

Cleveland Institute of Engineers, Middlesbrough, England.

Colorado Association of Members, Am. Soc. C. E., H. J. Burt, Secy., 235 Equitable Building, Denver, Colo.

Engineers' and Architects' Club of Louisville, Ky., 303 Norton Building, Fourth and Jefferson Streets, Louisville, Ky.

Engineers' Club of Baltimore, Baltimore, Md.

Engineers' Club of Minneapolis, 17 South Sixth Street, Minneapolis, Minn.

Engineers' Club of Philadelphia, 1317 Spruce Street, Philadelphia, Pa.

Engineers' Club of St. Louis, 3817 Olive Street, St. Louis, Mo.

Engineers' Club of Toronto, 96 King Street, West, Toronto, Ont., Canada.

Engineers' Society of Pennsylvania, 219 Market Street, Harrisburg, Pa.

Engineers' Society of Western Pennsylvania, 803 Fulton Building, Pittsburg, Pa.

Institute of Marine Engineers, 58 Romford Road, Stratford, London, E., England.

Institution of Engineers of the River Plate, Buenos Aires, Argentine Republic.

Institution of Naval Architects, 5 Adelphi Terrace, London, W. C., England.

Junior Institution of Engineers, 39 Victoria Street, Westminster, S. W., London, England.

Koninklijk Instituut van Ingenieurs, The Hague, The Netherlands.

Louisiana Engineering Society, 321 Hibernia Bank Building, New Orleans, La.

Memphis Engineering Society, Memphis, Tenn.

Midland Institute of Mining, Civil and Mechanical Engineers, Sheffield, England.

Montana Society of Engineers, Butte, Montana.

North of England Institute of Mining and Mechanical Engineers, Newcastle-upon-Tyne, England.

Oesterreichischer Ingenieur- und Architekten-Verein, Eschenbachgasse 9, Vienna, Austria.

Pacific Northwest Society of Engineers, 803 Central Building, Seattle, Wash.

Rochester Engineering Society, Rochester, N. Y.

Sachsischer Ingenieur- und Architekten-Verein, Dresden, Germany.

Sociedad Colombiana de Ingenieros, Bogota, Colombia.

Societe des Ingenieurs Civils de France, 19 Rue Blanche, Paris, France.

Society of Engineers, 17 Victoria Street, Westminster, S. W., London, England.

Svenska Teknologforeningen, Brunkebergstorg 18, Stockholm, Sweden.

Tekniske Forening, Vestre Boulevard 18–1, Copenhagen, Denmark.

Western Society of Engineers, 1737 Monadnock Block, Chicago, Ill.

SEARCHES IN THE LIBRARY

In January, 1902, the Secretary was authorized to make searches in the Library, upon request, and to charge therefor the actual cost to the Society for the extra work required. Since that time many searches have been made, and bibliographies and other information on special subjects furnished.

The resulting satisfaction, to the members, who have made use of the resources of the Society in this manner, has been expressed frequently, and leaves little doubt that, if it were generally known to the membership that such work would be undertaken, many would avail themselves of it.

The cost is trifling compared with the value of the time of an engineer who looks up such matters himself, and the work can be performed quite as well, and much more quickly, by persons familiar with the Library.

In asking that such work be undertaken, members should specify clearly the subject to be covered, and whether references to general books only are desired, or whether a complete bibliography, involving search through periodical literature, is desired.

In reference to this work, the Appendix* to the Annual Report of the Board of Direction for the year ending December 31st, 1906, contains a summary of all searches made to that date.

LOCAL ASSOCIATIONS OF MEMBERS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

Colorado Association

(Abstract of Minutes of Meeting)

November 13th, 1909.—The regular meeting of the Colorado Association of Members, Am. Soc. C. E., was called to order at 8 P. M.; George G. Anderson, Vice-President, in the chair; H. J. Burt, Secretary, and present, also, 8 members and 7 guests.

The reading of the Minutes and other regular business was dispensed with, on account of the special character of the programme.

An address on "Work" was made by Mr. A. D. Parker, Vice-President of the Colorado and Southern Railroad Company, and the subject was discussed by a number of those present.

A resolution of thanks was unanimously voted to Mr. Parker.

Adjourned.

* *Proceedings*, Vol. XXXIII, p. 20 (January, 1907).

ACCESSIONS TO THE LIBRARY

(From November 9th to December 6th, 1909)

DONATIONS*

CYRUS HALL McCORMICK, HIS LIFE AND WORK.

By Herbert N. Casson. Cloth, 8¼ x 5¼ in., illus., 12 + 264 pp. Chicago, A. C. McClurg & Co., 1909. $1.50 net.

In the author's opinion anyone who wishes to understand the making of the United States must read the life of Cyrus Hall McCormick, which spanned the heroic period of the industrial advancement of the United States and represented the replacing of muscle with machinery, and the establishment of better ways and better times in farm and city alike. The Contents are: The World's Need of a Reaper; The McCormick Home; The Invention of the Reaper; Sixteen Years of Pioneering; The Building of the Reaper Business; The Struggle to Protect Patents; The Evolution of the Reaper; The Conquest of Europe; McCormick as a Manufacturer; Cyrus H. McCormick as a Man; The Reaper and the Nation; The Reaper and the World; Give Us This Day Our Daily Bread; Index.

CONCRETE INSPECTION.

A Manual of Information and Instructions for Inspectors of Concrete Work, with Standard and Typical Specifications. By Charles S. Hill. Cloth, 6 x 3½ in., illus., 8 + 179 pp. Chicago and New York, The Myron C. Clark Publishing Co.; London, B. & F. N. Spon, Ltd., 1909. $1.00 net.

This volume consists, it is stated, of a series of rules and directions to be followed in inspecting concrete work, with brief explanations of the reasons for each rule and of their importance. The author believes that the subject-matter is based on sound practice and acknowledged authority. The Contents are: Inspection of Concrete Materials; Inspection of Proportioning and Mixing; Inspection of Form Work; Inspection of Reinforcement; Inspection of Concreting; Inspection of Sidewalk Construction; Inspection of Molding and Driving Concrete Piles; Inspection of Cast Concrete Work; Standard and Typical Specifications; Index.

A TEXT BOOK ON GRAPHIC STATICS.

By Charles W. Malcolm, Assoc. M. Am. Soc. C. E. Cloth, 9 x 6 in., 12 + 316 pp. New York and Chicago, The Myron C. Clark Publishing Co.; London, E. & F. N. Spon, Ltd., 1909. $3.00 net.

It has been the aim of the author to give proofs of the principles stated, with full explanations of the construction. No attempt has been made, however, to give elaborate solutions which have little or no practical application. Most of the material in Parts I and II, the author states, has been used in printed form by his students for several years, and he expresses the hope that Parts II, III, and IV will be found of assistance to the practicing engineer. The Contents are: Part I, General Principles—Definitions; Concurrent Forces; Non-Concurrent Forces; Moments; Center of Gravity of Areas; Moment of Inertia. Part II, Framed Structures, Roof Trusses—Definitions; Loads; Reactions; Stresses in Roof Trusses; Wind Load Stresses; Stresses in Cantilever and Unsymmetrical Trusses—Maximum Stresses; Counterbracing; Three-Hinged Arch; Stresses in a Transverse Bent of a Building; Miscellaneous Principles. Part III, Beams—Bending Moments, Shears, and Deflections in Beams for Fixed Loads; Maximum Bending Moments and Shears in Beams for Moving Loads. Part IV, Bridges—Types of Bridge Trusses; Loads; Stresses in Trusses Due to Uniform Loads; Influence Diagrams, and Positions of Engine and Train Loads for Maximum Moments, Shears, and Stresses; Maximum Moments, Shears, and Stresses Due to Engine and Train Loads; Index.

SOLID BITUMENS:

Their Physical and Chemical Properties and Chemical Analysis, Together With a Treatise on the Chemical Technology of Bituminous Pavements. By S. F. Peckham. Cloth, 9 x 6 in., illus., 8 + 324 pp.

* Unless otherwise specified, books in this list have been donated by the publisher.

New York and Chicago, The Myron C. Clark Publishing Co.; London, E. & F. N. Spon, Ltd., 1909. $5.00 net.

This book, states the author, represents personal experiences and experiments prosecuted almost continuously for forty years by nearly every method of investigation proposed in the United States and Europe, and the reading of nearly everything published relating to bitumens, from Boussingault's Memoir to the present time. The author's aim has been to offer everything of value relating to the subject, and reference has been made to original memoirs as far as possible. The Contents are: Part I, The Natural History of Bitumens—Introduction; Geographical Distribution of Bitumens; Origin of Bitumens; The Classification of Bituminous Substances; The Derivation of Natural Solid Bitumens; The Derivation of Bituminous Rocks; The Derivation of Artificial Solid Bitumens. Part II, The Chemistry of Bitumens—General Considerations Concerning the Chemistry of Solid Bitumens; Boussingault's Memoir Upon the Composition of Bitumens; The Use of the Words, Petrolene, Asphaltene, Etc.; The Ultmate Analysis of Solid Bitumens; The Proximate Analysis of Solid Bitumens; The Technical Analysis of Solid Bitumens; Special Chemical and Physical Methods of Analysis by Which Solid Bitumens may be Recognized and Distinguished; Miscellaneous Methods Applied to Street Mixtures and Surfaces, Bituminous Paving Blocks, Bituminous Concrete, Wood Paving Blocks, Cements, Cement Mortars and Concrete, Etc. Part III, The Physical Properties of Solid Bitumens—Specific Gravity, Etc. Part IV, Chemical Technology of Bituminous Streets—Historical Introduction; A Modern Street; Bitulithic Streets; The Oiled Roads and Streets of the Pacific Coast; Asphalt Blocks; Wood Blocks; Conclusions; Index of Subjects; Index of Names.

LAVORAZIONA E TEMPERA DEGLI ACCIAI.

Indurimento Superficiale del Ferro e Cementazione. By Arturo Massenz. Seconda Edizione. Cloth, 6 x 4 in., illus., 16 + 118 pp. Milano, Ulrico Hoepli, 1909. 2 lira.

It has been the author's purpose to produce a popular book, written by a practical person, which will be an aid, especially to the worker of metals, in the method of forging and tempering steel. This second edition has been enlarged by filling in the omissions in the first, and by the addition of a special study on tempering furnaces, etc. A partial list of contents follows: Che cosa è l'acciaio?; Esigenze alle quali dovranno rispondere gli utensili da construire; Qualità d'acciaio preferirsi; Cambiamenti ai quali andrà soggetto l'acciaio nelle varie lavorazioni; Regole onde evitare gl'insuccessi nella tempera; Ricottura dopo la tempera; Cause principali determinanti gl'insuccessi nella lavorazione e nella tempera; Forno speciale per tempera, con bagno riscaldato elettricamente; Trattamento degli acciai secondo lo scopo pel quale sono destinati; Saldatura dell' acciaio, ecc.; Indurimento superficiale del ferro; Cementazione.

ELECTRIC POWER CONDUCTORS.

By William A. Del Mar. Cloth, 7¾ x 5¼ in., illus., 6 + 339 pp. New York, D. Van Nostrand Company, 1909. $2.00 net.

This book is intended to present a clear account of all engineering conditions which affect the purchase and use of power conductors. The author states that it will be found practical and up to date, being based on notes prepared for his own use, and containing nothing from any published data which has not been thoroughly studied and found to be reliable. The text is made as brief as possible, it is declared, and where explanation or theoretical discussion is advisable, it is supplemented by appendices. The sections on Alternating Current Feeder Calculations and Stress in Spans are written by Dr. Harold Pender. The first nine chapters pertain to wires and cables. The Contents are: Materials and Gauges; Electrical Properties; Insulation and Insulated Conductors; Determination of Size for Given Voltage Drop and Power Loss; Determination of Size for Given Stress in Spans; Specifications; Testing Wire and Cable; Installation; Depreciation and Deterioration; Third Rail Circuits; Rail Bonds; Tables of Inductance, Reactance and Capacity. Appendices: I, Basis of B. & S. Gauge; II, Basis of Skin Effect of Carrying Capacity Formulæ; III, Method of Calculating Thickness of Rubber Insulation; IV, Basis of Direct and Alternating Current Transmission; V, Basis of Formulæ for Stresses in Spans; VI, Explanation of Specifications; VII, Basis of Tables of Inductance; Index.

THE COLLECTION AND DISPOSAL OF MUNICIPAL WASTE.

By William F. Morse. Cloth, 9¼ x 6 in., illus., 17 + 462 pp. New York, The Municipal Journal and Engineer, 1909. $5.00.

In this work the author presents, it is stated, in as compact a form as possible, data gathered by him during nearly twenty-one years of continuous work, together

with information collected from scattered reports, papers, and a great variety of other sources. It is the author's purpose to give an historical sketch of disposal works in North American communities from the time the subject first assumed general importance (about 1885) to date. It is his further purpose to present an account of the various methods of waste collection and disposal in use in the United States and a comparison of the older with the more modern systems. There is also a short account of the progress of refuse disposal in other countries by W. Francis Goodrich, with short articles on "The Disposal of City Waste by Gas Producers," by F. C. Tryon, and "The Utilization of City Ashes," by C. H. Koyl. The book is divided into five parts, as follows: Part I, The Municipal Waste of American Towns; Part II, The Disposal of American Municipal Waste by Crematories and Incinerators; Part III, The Disposal of Waste by British Destructor Systems; Part IV, The Disposal of Waste by Reduction and Extraction Process; Part V, The Utilization of Municipal Waste.

RADIATION, LIGHT AND ILLUMINATION.

A Series of Engineering Lectures Delivered at Union College. By Charles Proteus Steinmetz. Compiled and Edited by Joseph Le Roy Hayden. Cloth, 9½ x 6 in., illus., 12 + 305 pp. New York, McGraw-Hill Company, 1909. $3.00.

The author states that these lectures are intended, not merely as a textbook on illuminating engineering or on the physics of light and heat radiation, but as an exposition, from the engineering point of view, of that knowledge of light and radiation which every educated man should possess. For this purpose they have been given in such form as to need no special knowledge of mathematics or engineering, and they describe the phenomena in plain language, with the exception of Lectures X and XI, which, by their nature, are somewhat mathematical, and are intended more particularly for the illuminating engineer. It is also stated that the lectures have been revised to date, and the important results of the National Bureau of Standards, contained in its recent bulletins, have been fully utilized. The compiler is authority for the statement that, in describing the experiments, numerical and dimensional data have been given, and the illustrations have been drawn to scale as far as possible, so as to make the repetition of the experiments convenient. The Contents are: Nature and Different Forms of Radiation; Relation of Bodies to Radiation; Physiological Effects of Radiation; Chemical and Physical Effects of Radiation; Temperature Radiation; Luminescence; Flames as Illuminants; Arc Lamps and Arc Lighting; Measurement of Light and Radiation; Light Flux and Distribution; Light Intensity and Illumination; Illumination and Illuminating Engineering; Physiological Problems of Illuminating Engineering; Index.

THE TECHNICAL PRESS INDEX, JANUARY, 1908, TO JUNE, 1909, INCLUSIVE.

Compiled in Cumulative Form from the Technical Press Index Published Monthly in *The Engineering Digest.* Cloth, 10 x 7¼ in., 24 + 373 pp. New York, The Technical Literature Company; London, Archibald Constable & Company, Limited, 1909. $3.00 net. (Donated by the Engineering News Publishing Co.)

The preface states that the items taken from the monthly index of *The Engineering Digest* have been rearranged, reclassified, and cross-indexed, for this first cumulative volume of the Technical Press Index, and that it is believed to contain reference to every important paper and article published, during the time covered, in the regular engineering periodicals of the United States, Canada, and Europe, and in the proceedings of the engineering and technical societies. All articles of evident temporary, local, or individual interest have been omitted, it is said, and an effort has been made to cover the field of engineering science in a permanently valuable manner. Where ambiguous titles have been used by authors, or where amplification seemed desirable, descriptive notes have been added, and in every case the number of words, number of figures or illustrations, and price are given. Beside the cross-indexing there has been introduced also a key, or index to the Index, consisting of an alphabetical list of technical catch-words or key-words, giving the different pages on which a subject will be found, many of the topics being brought out under several different heads. The main divisions, or heads, are as follows: Architecture; Automobiles and Aerial Navigation; Civil Engineering; Economics and Education; Electrical Engineering; Industrial Technology; Marine Engineering; Mechanical Engineering; Metallurgy; Mining Engineering; Municipal Engineering; Railway Engineering.

Gifts have also been received from the following:

Am. Ry. Master Mechanics Assoc. 1 bound vol.
Arnold, B. J. 1 bound vol.
Assoc. of Licensed Automobile Mfrs. 1 pam.
Bartholdt, Richard. 1 pam.
Boston, Mass.-Transit Comm. 1 bound vol.
Camden, N. J.-Chf. Engr. of the Water Dept. 1 pam.
Canada-Board of Ry. Commrs. 1 pam.
Canada-Dept. of Marine and Fisheries. 1 vol., 2 pam.
Canada-Geol. Survey Branch. 4 pam.
Cincinnati Univ. 1 pam.
Committee of One Hundred on National Health. 1 pam.
Conway, George R. G. 1 pam.
Dist. of Columbia-Engr. Comm. 2 vol.
Eng. Assoc. of the South. 1 pam.
Fairmount Park Art Assoc. 8 pam.
Fanning, John T. 1 vol.
Fendall, Benjamin T. 8 bound vol.
Fidelity & Casualty Co. of New York. 2 pam.
Hering & Fuller. 1 pam.
Illinois-Bureau of Labor Statistics. 1 bound vol.
Illinois-R. R. and Warehouse Comm. 1 bound vol.
Illinois-State Geol. Survey. 1 bound vol.
Illinois-Water Supply Assoc. 1 bound vol.
Incorporated Assoc. of Municipal and County Engrs. 1 bound vol.
Institution of Civil Engrs. 1 bound vol.
Iowa-Board of R. R. Commrs. 1 bound vol.
Joint Committee on Conservation. 1 pam.
Langthorn, Jacob E. 1 pam.
Lehigh & Hudson River Ry. Co. 1 pam.
McMinn, Thomas J. 1 bound vol., 1 vol.
Manchester Assoc. of Engrs. 2 bound vol.
Master Car Builders' Assoc. 1 bound vol.
Maury, Dabney H. 2 pam.
Mich.-R. R. Comm. 1 bound vol.
Mobile & Ohio R. R. Co. 1 pam.
Montreal, Canada-City Clerk. 1 bound vol.
Morris, Marshall. 1 bound vol.
National Elec. Light Assoc. 3 bound vol.
New York City-Board of *City Record*. 4 bound vol.
New York City-Board of Health. 1 bound vol.
New York City-Dept. of Docks and Ferries. 1 bound vol.
New York State-Commrs. of the State Reservation at Niagara. 1 pam.
New York State-Public Service Comm., First Dist. 4 pam.
Norboe, P. M. 1 pam.
North Dakota-State Geol. Survey. 2 vol.
North-East Coast Institution of Engrs. and Shipbuilders. 1 bound vol.
Ohio Elec. Light Assoc. 1 bound vol.
Philadelphia, Pa.-Bureau of Surveys. 1 bound vol.
Philadelphia, Pa.-Bureau of Water. 2 vol.
Pratt Inst. Free Library. 1 pam.
Roberts & Abbott Co. 2 pam.
Rogers, J. W. 1 pam.
Sanderson & Porter. 1 pam.
South African Assoc. of Engrs. 1 vol.
South Australia-Rys. Commr. 1 pam.
Southern Pacific Co. 2 pam.
Svenska Teknologföreningen. 1 pam.
Syracuse, N. Y.-Intercepting Sewer Board. 1 pam.
Union Univ. 1 pam.
U. S.-Chf. of Engrs. 3 bound vol., 6 pam.
U. S.-Coast and Geodetic Survey. 2 pam.
U. S.-Geol. Survey. 9 pam.
U. S.-Interstate Commerce Comm. 4 pam.
U. S.-Isthmian Canal Comm. 1 vol.
U. S.-Lake Survey Office. 1 pam., 1 map.
U. S.-Office of Exper. Stations. 6 pam.
U. S.-War Dept. 1 bound vol.
Victoria, Australia-Chf. Engr. for Ry. Constr. 1 pam.
Victoria, Australia-State Rivers and Water Supply Comm. 1 pam.
Western Australia-Commr. of Rys. 1 pam.
Williams, Winfield S. 2 pam.

BY PURCHASE

Minutes of Seventh Annual Meeting of the National Conference on Standard Electrical Rules, held at the Engineering Building, 29 West 39th Street, New York City, March 26th, 1909.

The Water Works Directory and Statistics, 1909. Including Index of Officials and List of Associations of Water Engineers. Hazell, Watson & Viney, Ltd., London, 1909.

American Inland Waterways. Their Relation to Railway Transportation and to the National Welfare; Their Creation, Restoration and Maintenance. By Herbert Quick. G. P. Putnam's Sons, New York and London, 1909.

Allgemeine Theorie über die veränderliche Bewegung des Wassers in Leitungen. I. Teil: Rohrleitungen, von Lorenzo Allièvi (Deutsche, erläuterte Ausgabe, bearbeitet von Robert Dubs und V. Bataillard). II. Teil: Stollen und Wasserschloss, von Robert Dubs. Julius Springer, Berlin, 1909.

Die Berechnung von Steifrahmen nebst Anderen Statisch Unbestimmten Systemen. Von Ejnar Björnstad. Julius Springer, Berlin, 1909.

Transactions of the Institution of Mining and Metallurgy. General and Personal Index, Vols. I to XV, 1892–1906. E. & F. N. Spon, Limited, and the Institution of Mining and Metallurgy, London; Engineering and Mining Journal, New York, 1909.

SUMMARY OF ACCESSIONS

(From November 9th to December 6th, 1909)

Donations (including 17 duplicates)	143
By purchase	6
Total	149

MEMBERSHIP

ADDITIONS

(November 10th to December 7th, 1909)

MEMBERS		Date of Membership.
ARTHUR, HOWARD ELMER. Chf. Engr., Mosher Mfg. Co., Dallas, Tex		Nov. 8, 1909
DENCER, FREDERICK WILLIAM. Engr., Lassig Plant, Am. Bridge Co., Wrightwood and Clybourn Aves., Chicago, Ill	Assoc. M. M.	Feb. 7, 1906 Nov. 8, 1909
ELLERY, NATHANIEL. State Engr., Dept. of Eng., Sacramento, Cal		Nov. 8, 1909
GODFREY, EDWARD. Structural Engr., Robert W. Hunt & Co., Monongahela Bank Bldg., Pittsburg, Pa		Oct. 5, 1909
GREGORY, WILLIAM BENJAMIN. Prof. of Experimental Eng., Tulane Univ, and Irrig. Engr., U. S. Dept. of Agri., New Orleans, La		Nov. 8, 1909
HAZEN, WILLIAM NELSON. Engr., Expanded Metal Eng. Co., Union Bldg., Newark, N. J	Assoc. M. M.	Dec. 3, 1902 Nov. 8, 1909
KNOX, SCHUYLER BRUSH. Eastern Mgr. and Contr. Engr., Fort Pitt Bridge Works, 45 Broadway, New York City		Nov. 8, 1909
LALL, CHIRANJI. Head Master, Govt. School of Eng., Anarkali, Nila Gumbad, Mool Chand New Bldg., Lahore, Punjab, India		Oct. 5, 1909
LAYTON, HUDSON FLACK. Civ. and Cons. Engr., 413 Fourth Ave., Pittsburg, Pa		Nov. 30, 1909
LORRAINE, MADISON JOHNSON. 2107 Dwight Way, Berkeley, Cal		Nov. 8, 1909
MCKAY, GEORGE ALBERT. Civ. Engr., U. S. N., U. S. Naval Training Station, Great Lakes, North Chicago, Ill	Assoc. M. M.	Feb. 4, 1903 Nov. 8, 1909
MORRIS, CLYDE TUCKER. Prof. of Structural Eng., Ohio State Univ., Columbus, Ohio.	Assoc. M. M.	Mar. 6, 1907 Nov. 8, 1909
MURALT, CARL LEONARD DE. Prof. of Elec. Eng., Univ. of Michigan; Cons. Engr. (Muralt & Co.), 114 Liberty St., New York City		Aug. 31, 1909
NICHOLS, CHARLES HART. 42 East 23d St., New York City.		Nov. 8, 1909
REYNOLDS, ABRAHAM MOREAU. Engr. and Supt., Essex County Park Comm., 60 Clifton Ave., Newark, N. J.		Nov. 30, 1909
SCHERER, CLINTON LYTER. City Engr., Beaumont, Tex		Nov. 8, 1909
SCHNEIDER, EDWARD JOHN. Contr. Mgr., Am. Bridge Co. of New York, 906 Crocker Bldg., San Francisco, Cal	Assoc. M.	July 9, 1906 Nov. 8, 1909
SMITH, FRANCIS PITT. Chemical and Cons. Paving Engr. (Dow & Smith), 24 East 21st St., New York City		Nov. 30, 1909

MEMBERS (*Continued*).		Date of Membership.
VAN WINKLE, EDWARD. Cons. Patent Engr., The West Street Bldg., New York City		Nov. 30, 1909
WADSWORTH, GEORGE REED. Cons. Engr., 19 Congress St., Boston, Mass	Assoc. M. M.	Nov. 7, 1906 Nov. 8, 1909
WOOLDRIDGE, CHARLES LAWSON. Superv. Engr., T. A. Gillespie Co., 125 South Lang Ave., Pittsburg, Pa		Oct. 5, 1909
YUILLE, NATHANIEL ALSTON. U. S. Asst. Engr., Box 709, Mobile, Ala	Assoc. M. M.	June 6, 1906 Nov. 8, 1909
ASSOCIATE MEMBERS		
ARCE, JULIUS ANDREW. Apartado 90, Arequipa, Peru		Aug. 31, 1909
ARMSTRONG, ROGER WELLINGTON. Asst. Designing Engr., N. Y. Board of Water Supply, 299 Broadway, New York City		Nov. 8, 1909
ATTERBURY, CHARLES DE LA PLANE. City Engr., Centralia, Lewis Co., Wash		Nov. 8, 1909
BAETA-NEVES, LOURENCO. Chf. Engr., Technical Dept. of Railways, Public Works and Industries. Bello Horizonte, Minas Geraes, Brazil		Nov. 30, 1909
BARNARD, WILFRED KEEFER. Asst. Engr., M. of W., San Pedro, Los Angeles & Salt Lake R. R., 2711 Ellendale Pl., Los Angeles, Cal		Nov. 8, 1909
BEALE, HARRY ORLANDO. Care, U. S. Reclamation Service, Fort Shaw, Mont		Oct. 5, 1909
BERRY, FRANCIS RIGDON. Care, Portsmouth, Berkley & Suffolk Water Co., Portsmouth, Va	Jun. Assoc. M.	Dec. 2, 1902 Nov. 8, 1909
BLACK, RALPH PETERS. Div. Engr., K. & M. Ry., Charleston, W. Va		Nov. 8, 1909
BLOOD, CHARLES FREDERICK. Ridgefield, N. J		Oct. 5, 1909
BOWIE, CLIFFORD PINKNEY. Asst. Engr., Associated Pipe Line Co., 1360 Spring St., Berkeley, Cal		Aug. 31, 1909
BOYD, BUTLER BENNETT. Asst. Engr., San Diego & Arizona Ry., 3210 F St., San Diego, Cal		Nov. 8, 1909
COLLINS, FRANK JOSEPH. 600 E. Fayette St., Syracuse, N. Y		Nov. 8, 1909
DAVIES, WILLIAM GOMER. Willows, Cal	Jun. Assoc. M.	Feb. 2, 1904 Nov. 8, 1909
DAVIS, BENJAMIN HERMAN. Asst. Engr., D., L. & W. R. R., Hoboken, N. J		Nov. 30, 1909
DUDER, JOHN. Asst. City Engr., 1080 Third Ave., Salt Lake City, Utah		Nov. 8, 1909
EVANS, JOHN EDWARD. Engr. of Alignment, Detroit River Tunnel Co., 84 Trumbull Ave., Detroit, Mich		Nov. 8, 1909
GOODMAN, LOUIS. Pres., The Wormser-Goodman Constr. Co., 2 Rector St., Room 520, New York City		Oct. 5, 1909
HASBROUCK, OSCAR. 136 Saratoga Ave., Northside, Cohoes, N. Y	Jun. Assoc. M.	Jan. 3, 1907 Nov. 8, 1909

ASSOCIATE MEMBERS (*Continued*).	Date of Membership.
HAWN, RUSSELL JOHN. Supt., Virginia Portland Cement Co., Fordwick, Va.	Nov. 30, 1909
HAZARD, WILLIAM ABBOTT. Mgr., Lackawanna Bridge Co., Buffalo, N. Y.	Nov. 8, 1909
HIGGINSON, JONATHAN YATES. Res. Engr., N. Y., N. H. & H. R. R., 88 Centre St.,	Jun. Oct. 1, 1901
New Rochelle, N. Y.	Assoc. M. Nov. 8, 1909
JOHNSTON, CHARLES EUGENE. Div. Engr., Kansas City Southern Ry., Mena, Polk Co., Ark.	June 1, 1909
JONES, FREDERICK ARMISTEAD. Res. Engr., Mo. Pac. Ry. Co., Ft. Gibson, Okla.	Nov. 8, 1909
LONG, CLARENCE BURTON. Augusta, Lewis and Clark County, Mont.	Nov. 8, 1909
MAUPIN, EDGAR STAPLES. Jun. Engr., U. S.	Jun. Sept. 5, 1905
Engr. Office, Champagnolle, Ark.	Assoc. M. Nov. 8, 1909
MAYHEW, ALFRED BOARDMAN. Asst. Engr., U. S. Reclamation Service, Boise, Idaho.	Nov. 8, 1909
MINER, JAMES HENRY. Asst. Engr., U. S.	Jun. Mar. 31, 1903
Reclamation Service, Montrose, Colo.	Assoc. M. Nov. 8, 1909
MITCHAM, GEORGE NATHAN. Prof. of Civ. Eng., Alabama Poly. Inst., Auburn, Ala.	Nov. 8, 1909
MORROW, FREDERICK EDGAR. 181 La Salle St., 10th Floor, Chicago, Ill.	Nov. 8, 1909
NEWMAN, ARTHUR TRAVER. Care, A. L. Webster, 82 Wall St., New York City.	Nov. 8, 1909
NORCROSS, THEODORE WHITE. Asst. Engr., U. S. Geological Survey, Water Resources Branch, 6 Beacon St., Room 809, Boston, Mass.	Nov. 8, 1909
PLOCK, ALBERT FRANCIS. Chf. Engr., Northwestern Iron Co., Mayville, Wis.	Nov. 8, 1909
PRATT, HENRY BLANCHARD. Asst. Engr., J. R. Worcester & Co.; Res., 750 Lexington St., Waltham, Mass.	June 1, 1909
ROBBINS, HALLET RICE. Civ. and Min. Engr.,	Jun. Nov. 5, 1907
P. O. Box 51, Seattle, Wash.	Assoc. M. Nov. 8, 1909
RUSCH, HENRI. Chf. Engr., Bldg. Dept., 770 Bayard Ave., St. Louis, Mo.	Nov. 8, 1909
SANFORD, WALTER EDWARD. Asst. Engr., P. T. & T. R. R. Co., Hammond Pl., Elmhurst, N. Y.	Nov. 30, 1909
SMITH, FORREST LEIGH. Township Engr., 102	Jun. April 30, 1907
Smith St., Perth Amboy, N. J.	Assoc. M. Nov. 8, 1909
SPEARMAN, CHARLES. Room 210, Boise City National Bank Bldg., Boise, Idaho.	Oct. 5, 1909
SPENCER, LOUIS BERNARD. Civ. and Min.	Jun. Oct. 31, 1899
Engr., P. O. Box 499, Ogden, Utah.	Assoc. M. Nov. 8, 1909
STRATHMANN, EDWARD CHARLES. Gen. Supt., Bedford Stone & Constr. Co., 814 Traction Bldg., Indianapolis, Ind.	Nov. 8, 1909

ASSOCIATE MEMBERS (*Continued*).

	Date of Membership.
THOMAS, WILLIAM MICHAEL. 1012 Union Trust Bldg., Los Angeles, Cal.	Nov. 8, 1909
UNGER, ERIK BIRGER LUND. Mexican R. R., City of Mexico, Mexico.	Nov. 8, 1909
WALKER, EDWARD MANSFIELD. Engr. of Alignment, Detroit River Tunnel Co., 489 Fort St., West, Detroit, Mich.	Nov. 8, 1909
WHITE, DAVID MILLER. P. O. Box 278, Midland, Tex.	Nov. 8, 1909
WHITING, GEORGE WILLIAM CARLYLE. Asst. Engr., Baltimore Sewerage Comm., 112 E. Preston St., Baltimore, Md. — Jun.	Sept. 5, 1905
— Assoc. M.	Nov. 8, 1909
WILLIAMS, LESTER DENNISON. 702½ Marshall St., Milwaukee, Wis.	Nov. 8, 1909
WOOD, ROBERT WALTER. Asst. Engr. in Chg. of Port Richmond Div., Borough of Richmond, New York City, 13 Van Pelt Ave., Mariner Harbor, N. Y. — Jun.	April 30, 1907
— Assoc. M.	Nov. 8, 1909

ASSOCIATES

CUSHMAN, ALLERTON SEWARD. Asst. Director, Office of Public Roads, Dept. of Agriculture, 1314 Sixteenth St., N. W. Washington, D. C.	Nov. 8, 1909
STRUCKMANN, HOLGER. Third Vice-Pres., Union Sand & Material Co., 6330 Washington Bldg., St. Louis, Mo.	Nov. 30, 1909

JUNIORS

BLUHM, HERMAN WILLIAM. Draftsman, Isthmian Canal Comm., Corozal, Canal Zone, Panama.	Nov. 8, 1909
BOCK, CARL AUGUST. Casilla 43, Oruro, Bolivia.	Aug. 31, 1909
BRAINERD, HAROLD AFFLECK. 360 W. Church St., Elmira, N. Y.	Nov. 30, 1909
CARPENTER, J. C. City Engr. and Surv., Watertown, S. Dak.	Nov. 8, 1909
CHANDLER, EMERSON LAWRENCE. 16 Squire St., New London, Conn.	Nov. 8, 1909
CLIFFORD, WALTER WOODBRIDGE. 60 Oak St., Hyde Park, Mass.	Nov. 8, 1909
DUNAN, GEORGE EDMUND. Asst. Engr., Morey-Faulhaber Constr. Co., Apalachicola, Fla.	Nov. 8, 1909
GRAHAM, GERMAIN PAUL. 280 Quail St., Albany, N. Y.	Nov. 8, 1909
HAYWOOD, CHARLES ELLSWORTH. 37 Van Name Ave., Mariner Harbor, N. Y.	Nov. 8, 1909
HESS, JOHN STRIDER. Box 967, Dawson, Y. T., Canada.	Oct. 5, 1909
LEONARD, OLIVER YEATON. Instrumentman, Porto Rico Irrig. Service, Guayama, Porto Rico.	Nov. 8, 1909
LILLY, RIDGELY CASEY. U. S. Inspector, 3d Dist., Mississippi River Comm., Box 404, Vicksburg, Miss.	Oct. 5, 1909
MISCHKE, ALEXANDER. Jouravlinaya 16, Warsaw, Russia.	Oct. 5, 1909

JUNIORS (*Continued*).	Date of Membership.
SCHOBINGER, GEORGE. 181 La Salle St., Chicago, Ill.....	Oct. 5, 1909
SCOTT, WALTER VANDERBELT. 511 Gifford Pl., Hudson, N. Y.	Nov. 8, 1909
SHANKLAND, RALPH GRAHAM. Supt. of Concrete Constr. for E. C. & R. M. Shankland, 1106 The Rookery Bldg., Chicago, Ill.....	Nov. 8, 1909
SNYDER, HUNTER IMBODEN. 21 Cotton Exchange Bldg., Memphis, Tenn.....	Nov. 30, 1909
STEWART, WALTER PHELPS. U. S. Engr. Office, Nashville, Tenn.....	Nov. 8, 1909
SWENSSON, OTTO JORDAN. Office, Board of Water Supply Yorktown Heights, N. Y.....	Nov. 8, 1909
TATUM, ROBERT LEE. Benoit, Miss.....	Nov. 8, 1909
WINANS, LAWRENCE LEWIS. County Surv., Audrain Co., Mexico, Mo.....	Nov. 8, 1909

RESIGNATION

ASSOCIATE	Date of Resignation.
LOWE, HOUSTON.....	Nov. 8, 1909

DEATHS

BALL, ERNEST STEARNS. Elected Junior, May 2d, 1905; date of death unknown.

HARRIS, CHARLES MARSHALL. Elected Associate, January 8th, 1873; died October 28th, 1909.

SANDERSON, JAMES GARDNER. Elected Member, July 6th, 1881; died November 15th, 1909.

Total Membership of the Society, December 7th, 1909, 5 273.

MONTHLY LIST OF RECENT ENGINEERING ARTICLES OF INTEREST

(November 9th to December 6th, 1909)

NOTE.—*This list is published for the purpose of placing before the members of the Society, the titles of current engineering articles, which can be referred to in any available engineering library, or can be procured by addressing the publication directly, the address and price being given wherever possible.*

LIST OF PUBLICATIONS

In the subjoined list of articles, references are given by the number prefixed to each journal in this list:

(1) *Journal*, Assoc. Eng. Soc., 31 Milk St., Boston, Mass., 30c.
(2) *Proceedings*, Engrs. Club of Phila., 1317 Spruce St., Philadelphia, Pa.
(3) *Journal*, Franklin Inst., Philadelphia, Pa., 50c.
(4) *Journal*, Western Soc. of Engrs., Monadnock Blk., Chicago, Ill.
(5) *Transactions*, Can. Soc. C. E., Montreal, Que., Canada.
(6) *School of Mines Quarterly*, Columbia Univ., New York City, 50c.
(7) *Technology Quarterly*, Mass. Inst. Tech., Boston, Mass., 75c.
(8) *Stevens Institute Indicator*, Stevens Inst., Hoboken, N. J., 50c.
(9) *Engineering Magazine*, New York City, 25c.
(10) *Cassier's Magazine*, New York City, 25c.
(11) *Engineering* (London), W. H. Wiley, New York City, 25c.
(12) *The Engineer* (London), International News Co., New York City, 35c.
(13) *Engineering News*, New York City, 15c.
(14) *The Engineering Record*, New York City, 12c.
(15) *Railroad Age Gazette*, New York City, 15c.
(16) *Engineering and Mining Journal*, New York City, 15c.
(17) *Electric Railway Journal*, New York City, 10c.
(18) *Railway and Engineering Review*, Chicago, Ill., 10c.
(19) *Scientific American Supplement*, New York City, 10c.
(20) *Iron Age*, New York City, 10c.
(21) *Railway Engineer*, London, England, 25c.
(22) *Iron and Coal Trades Review*, London, England, 25c.
(23) *Bulletin*, American Iron and Steel Assoc., Philadelphia, Pa.
(24) *American Gas Light Journal*, New York City, 10c.
(25) *American Engineer*, New York City, 20c.
(26) *Electrical Review*, London, England.
(27) *Electrical World*, New York City, 10c.
(28) *Journal*, New England Water-Works Assoc., Boston, Mass., $1
(29) *Journal*, Royal Society of Arts, London, England, 15c.
(30) *Annales des Travaux Publics de Belgique*, Brussels, Belgium.
(31) *Annales* de l'Assoc. des Ing. Sortis des Ecoles Spéciales de Gand, Brussels, Belgium.
(32) *Mémoires et Compte Rendu des Travaux*, Soc. Ing. Civ. de France, Paris, France.
(33) *Le Génie Civil*, Paris, France.
(34) *Portefeuille Economiques des Machines*, Paris, France.
(35) *Nouvelles Annales de la Construction*, Paris, France.
(37) *Revue de Mécanique*, Paris, France.
(38) *Revue Générale des Chemins de Fer et des Tramways*, Paris, France.
(41) *Modern Machinery*, Chicago, Ill., 10c.
(42) *Proceedings*, Am. Inst. Elec. Engrs., New York City, 50c.
(43) *Annales des Ponts et Chaussées*, Paris, France.
(44) *Journal*, Military Service Institution, Governors Island, New York Harbor, 50c.
(45) *Mines and Minerals*, Scranton, Pa., 20c.
(46) *Scientific American*, New York City, 8c.
(47) *Mechanical Engineer*, Manchester, England.
(48) *Zeitschrift*, Verein Deutscher Ingenieure, Berlin, Germany.
(49) *Zeitschrift für Bauwesen*, Berlin, Germany.
(50) *Stahl und Eisen*, Düsseldorf, Germany.
(51) *Deutsche Bauzeitung*, Berlin, Germany.
(52) *Rigasche Industrie-Zeitung*, Riga, Russia.
(53) *Zeitschrift*, Oesterreichischer Ingenieur und Architekten Verein, Vienna, Austria.
(54) *Transactions*, Am. Soc. C. E., New York City, $4.
(55) *Transactions*, Am. Soc. M. E., New York City, $10.
(56) *Transactions*, Am. Inst. Min. Engrs., New York City, $5.

(57) *Colliery Guardian*, London, England.
(58) *Proceedings*, Engrs.' Soc. W. Pa., 803 Fulton Bldg., Pittsburg, Pa., 50c.
(59) *Transactions*, Mining Inst. of Scotland, London and Newcastle-upon-Tyne, England.
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Die Schliesse im Scheitrechten Ziegelgewölbe.* F. v. E. (78) Oct. 30.
Bewehrte Steildecken.* Emil Mogensen. (78) Oct. 30.
Versuche über die Tatsächliche Widerstandsfähigkeit von Balken mit ⊏-förmigen Querschnitt.* C. Bach. (48) Oct. 30.
Verhalten des Betons bei Niederer Temperatur.* (80) Nov. 6.
Eisenbetonkuppel- und Wölbekonstruktionen.* Karl W. Mautner. (Paper read before the Deutsche Betonverein.) (80) Nov. 20.

Water Supply.

Hydro-Electric Power in Canada.* Cecil B. Smith, M. Am. Soc. C. E. (54) Vol. 65.
The Waters of the Great Lakes. R. B. Dole. (28) Sept.
The Development of the Camaguey Water Works, Cuba.* Henry A. Young. (28) Sept.
The Poughkeepsie Water Works.* John C. Otis. (28) Sept.
Disinfection as an Adjunct to Water Purification. H. W. Clark and Stephen DeM. Gage. (28) Sept.
Odors and Tastes in the Water Supply of Holyoke.* James L. Tighe. (28) Sept.
The Ludlow Filters. Carroll F. Story. (28) Sept.
Water Supply in the Metropolitan Tower, New York.* (70) Nov.
Building the New Reservoir for New York's Water Supply. (67) Nov.
Waterproofing a Shaft of the Blue Island Ave. Water Tunnel at Chicago.* (13) Nov. 11.
The Standley Lake Dam, near Denver, Colorado.* (14) Nov. 13.
The Bernhart Sand Filters, Reading, Pa.* (14) Nov. 13.
Wallkill River Hydro-Electric Plant, Walden, New York. Harry W. Dennis. (Abstract from *Cornell Civ. Engr.*) (14) Nov. 13.
Torresdale Pumping Plant and Filters.* Thomas Wilson. (64) Nov. 16.
Some Detail Tunnel Costs in Tunnel No. 7 of the Los Angeles Aqueduct. C. H. Richards. (13) Nov. 18.
Methods and Cost of Working in the Red Rock Tunnel during a Month of Record Advance.* (13) Nov. 18.
Method of Calculating Horse-Power. Dennis Stovall. (Abstract from *Min. World.*) (96) Nov. 19.
The Water Supply System of Newton, Mass.* (14) Nov. 20.
Water Purification at Wilmington, Delaware.* (14) Nov. 20.
Huge Venturi Meters Registering 625 Cu. Ft. per Second, Divi Island Pumping Plant, Madras, India.* (13) Nov. 25.
The High Pressure Fire Protection System, Winnipeg. (14) Nov. 27.
A Collapsible Steel Dam at Garland, Utah.* (14) Nov. 27.
Wash-Drill Borings for the Standley Lake Dam. (14) Nov. 27.
The New Water Filtration Plant at Toledo, Ohio, with Data on Contract Methods and Cost.* B. M. Baker. (86) Dec. 1.
The Partial Failure through Undermining of the Zuni Dam, New Mexico.* (13) Dec. 2.
The East Orange Water Wells Decision. (14) Dec. 4.
Cost of Laying Water Mains at Cheyenne.* H. G. Porak. (14) Dec. 4.
Experiments on the Passage of Bacteria through Soil. Fritz Ditthorn and Arthur Luerssen. (Abstract from *Gesundheits-Ingenieur.*) (14) Dec. 4.
The Pipe Line of the Cañon City Water Supply System.* (14) Dec. 4.
Setting Heavy Steel Flood Gates.* (14) Dec. 4.
Note sur l'Irrigation de la Plaine de Konia (Asie Mineure). Godard. (43) Sept.
Ueber den Antrieb von Hubkolbenpumpen durch Turbinen. Hans Wettich. (97) Serial beginning Nov. 5.
Die Turbinenversuchsanstalten und die Wasserkraftwerke mit Wasserkraftspeicher der Firma J. M. Voith.* Fritz Oesterlen. (48) Serial beginning Nov. 6.

*Illustrated.

Waterways.

The Waters of the Great Lakes. R. B. Dole. (28) Sept.
The Pollution of the Merrimack River. (Abstract from Report of the Mass. State Board of Health.) (13) Nov. 11.
Transport across Sydney Harbour.* (11) Nov. 19.
The Work at Panama during the Last Fiscal Year.* (14) Nov. 27; (13) Nov. 25.
The King's Dock, Swansea.* (12) Serial beginning Nov. 19; (22) Nov. 26.
Lake Currents near Toronto.* F. W. Thorold. (96) Dec. 3.
The Improvement of the Mississippi River. Dickinson. (Abstract of report to the Deep Waterways Convention.) (14) Dec. 4.
Protecting Levee Slopes with Concrete.* (14) Dec. 4.
Die Schwimmerschleuse mit Riegeln und Bremsen.* Fr. Jebens. (81) Vol. 6, 1909.
Der Bau Zweiter Schleusen bei Wernsdorf und Kersdorf. (Spree-Oder-Wasserstrasse.)* Engelhard u. Zimmermann. (49) Vol. X.-XII., 1909.

*Illustrated.

REPORTS OF SPECIAL COMMITTEES.

Progress Report of the Special Committee on Bituminous Materials for Road Construction.

The Board of Direction,
American Society of Civil Engineers:

GENTLEMEN:

Your Special Committee, appointed May 4th, 1909, to report on "Bituminous Materials for Road Construction and on Standards for Their Test and Use," respectfully presents the following Progress Report.

On receiving notice of its appointment, your Committee organized by electing W. W. Crosby, Chairman, and A. H. Blanchard, Secretary. Your Committee agreed at once that the available information concerning its subject was so diversified in both form and character as to render it impossible of satisfactory use by the Committee, and the question of securing additional information on uniform lines was considered.

As actual construction had already begun in many instances, it seemed imperative to establish at the earliest possible moment a schedule of information concerning such work, which should cover all the points on which it appeared that information was necessary, and yet which should, if possible, not be so extensive in form as to deter anyone from attempting to respond to a large portion of the questions asked.

In preparing this schedule, the earlier reports from many sources were considered, and, as far as seemed to the Committee practicable, an effort was made to include in its schedule the points that seemed to others important. It was also found necessary, in order to secure uniformity, to suggest methods for arriving at some of the figures requested. As far as it seemed practicable, already standardized methods were adopted and only such changes in ordinary methods were inserted as seemed necessary for accuracy or improvement. In some instances new methods had to be devised.

The Committee submitted a list of analyses and methods of testing bituminous materials to many engineers and chemists, conversant with the subject, for criticism and suggestions. After considerable correspondence, and a series of meetings of the Committee, and after continued revision, the final list was adopted as representing most nearly the views of the majority.

A circular letter was then prepared, explaining the intentions of the Committee and inviting the co-operation and assistance of those interested in its work.

The letter and forms were printed and distributed about August 1st to National, State, County, Town and City Highway Engineers and Officials, and to Chemists, Manufacturers of, and Agents for, Bituminous Materials. More than 800 letters were sent out during August and September.

From replies already received, your Committee feels greatly encouraged by the interest manifested in its work, in spite of, as we recognize, the amount of work requested by it of its correspondents. The States of Maine, Massachusetts, Maryland, Pennsylvania, New Hampshire, New York, and Rhode Island have agreed to report to the Committee on the bituminous macadam roads constructed during 1909, including the analyses of the bituminous materials used, made in accordance with the methods suggested by the Committee.

At this date, your Committee can scarcely report more than "Progress." Conclusions cannot be drawn on most points until the results of the action of the varying seasons and traffic conditions shall have become apparent. It is hoped that partial conclusions may be justified in time to report at the next Annual Meeting.

The Committee begs leave to express here its profound appreciation of the courtesy and material assistance rendered it in its work by your Secretary, Mr. Charles Warren Hunt, and also by the engineers, chemists, and manufacturers with whom it has had the good fortune to come in contact, and to sign itself

Very respectfully,

W. W. Crosby, *Chairman.*
H. K. Bishop.
A. W. Dean.
A. H. Blanchard, *Secretary.*

Progress Report of the Special Committee on Steel Columns and Struts.

To the Members of the American Society of Civil Engineers in Annual Meeting:

The Special Committee authorized by vote of this Society "to consider and report upon the design, ultimate strength and safe working values of Steel Columns and Struts" submits the following report of progress.

Your Committee met and organized on January 6th, 1909. Since that time four meetings of the Committee as a whole and numerous sub-committee meetings have been held. Time and labor have been spent in collecting existing literature on the subject of tests of compression members, in investigating and compiling the data now accessible, in a preliminary consideration of the extent to which it is safe to draw conclusions from data now available, and in a study of the possibility of securing additional information. At the last Annual Meeting, your Committee submitted to the Society a resolution requesting the United States Government to construct a machine capable of testing to destruction compression members of large dimensions.

Your Committee finds that there are at present available for use in this country, but three testing machines of large size, while only two are capable of making compression tests. The data giving principal dimensions and capacity of the machines are submitted in the following table:

DATA RELATING TO TESTING MACHINES.

	Phœnixville, Pa.	Ambridge, Pa.	Watertown, Mass.
Date of completion	About 1886	1905	1879
Capacity for tension tests	2 600 000 lb.	4 000 000 lb.	500 000 lb.
Capacity for compression tests.	2 600 000 lb.	none	800 000 lb.
Maximum length of test pieces:			
Tension	50 ft.	39 ft.	20 ft.
Compression	55 "	none	26 " 3 in.
Available clear width for tests.	3 "	3 ft.	42½ in.
Power	Hydraulic	Hydraulic	Hydraulic
Recording device	Mercury gauges	Mercury gauge	Emery hydraulic scale
Height from floor to center of ram	About 4 ft.	2 ft.	47 in.

These machines will be supplemented in the near future by two others. A vertical machine of 10 000 000 lb. capacity in compression, capable of testing columns up to 65 ft. in length is now being built for the Geological Survey. This machine will be located in Pittsburg. Under authority of the Congress of 1908-09, the Bureau of Standards has contracted for a horizontal machine with a capacity of 2 300 000 lb.

in compression and 1 150 000 lb. in tension, of the same type that is now in use at Watertown Arsenal. This machine is to be located at Washington, D. C.

Your Committee has examined the records of numerous compression tests made on pieces varying in area of cross-section from laboratory specimens to 42 sq. in. While the results given by small specimen tests are interesting, your Committee has selected only the tests of columns large enough to be used as structural members, and these have been plotted on Figs. 1 to 22, inclusive. Fig. 23 gives a summary of results.* Your Committee has also considered some tests which it is not now at liberty to publish, and it is expected that the data given in this report will be supplemented from time to time as these and other tests become available for publication.

In the use of $\frac{l}{r}$ (length divided by radius of gyration) as a variable, your Committee realizes that r may not be the only variable affecting the strength of a column of given length. Shape, details, and dimensions, may make the results very different in columns having the same value of $\frac{l}{r}$. Until many extended series of tests afford more information with reference to these elements, it is necessary to follow common practice in plotting.

To reduce the results of the various series of tests shown on the diagrams to a uniform basis, the values have been adjusted to a grade of steel of 60 000 lb. ultimate tensile strength in specimen tests, by assuming that the ultimate compressive strength will vary as the ultimate tensile strength. This method of adjustment probably does not accurately represent the law of variation of the different grades of iron and steel, still, as the adjusted value is a fair medium between the older iron and the newer steel, it affords some basis for comparison.

A casual examination of the plates shows a meager range of experiments, made on no systematic plan, and a striking disagreement of results, even after adjustment for quality of material. It is evident that these data are not sufficient for the establishment of a satisfactory law for the strength of columns. This emphasizes the importance of further tests on columns arranged in series under conditions of material, workmanship and details, which will aid scientific observation in the establishment of the effects of the different variables.

At the time your Committee was appointed, circumstances caused a demand for tests to destruction of compression members equal in size to the largest ever built for existing bridges. This demand was backed largely by both the popular and technical press, but careful consideration and study lead to the conviction that while such tests are desirable,

*Not received in time to be reproduced with this report.

the cost of making them is so great that there is no chance of obtaining a sufficient number to disclose any fundamental law. Elaborate series of tests on smaller columns, in which the details and processes of manufacture conform to commercial practice, would disclose the effects of the different variables and would determine the fundamental laws of the behavior of columns under stress. Such a series of tests, with the necessary investigations, should properly be made by the Federal Government. For comparison with the results obtained from these series, tests of members, modeled after columns which are too expensive to allow more than an occasional test, and too large to be handled conveniently, would afford valuable data.

Your Committee recognizes the urgency of the demand for a statement giving values for ultimate strength and safe working stresses for compression members, but in their opinion existing data do not warrant the recommendation of any definite column formula. Until the subject can be studied more at length and until more extended series of tests have been made, your Committee does not deem it advisable to complicate the situation by offering a tentative formula which would vary but little from other formulas now in use. When the results of further tests become available for study and investigation, your Committee hopes to be able to make a recommendation for a column formula.

Committee.
Alfred P. Boller.
Austin Lord Bowman.
Emil Gerber.
Charles F. Loweth.
Ralph Modjeski.
Frank C. Osborn.
Geo. H. Pegram.
Lewis D. Rights.
Geo. F. Swain.
Emil Swensson.
Joseph R. Worcester.

For the Committee,

AUSTIN LORD BOWMAN,
Chairman.
LEWIS D. RIGHTS,
Secretary.

COLUMN TESTS.

Phoenix Iron Co. at Phoenixville, Pa., 1873.

Burr,- "Elasticity & Resistance of Materials."

Test No.	Material	Elastic Limit, Specimen Test	Ultimate Tension, Specimen Test.	Length, Inches.	Area, Sq. In.	Section.	$\frac{L}{R}$	Elastic Limit, Compression.	Ultimate Compression.
1	Iron.		46 000 - 48 000	285.7	5.84	○	141		30 274
2	"		" "	288.0	5.95		142		16 387
3	"		" "	280.5	10.21		95		36 419
4	"		" "	272.5	8.50		96		38 235
5	"		" "	278.4	13.31		96		32 742
6	"		" "	278.4	12.85		96		35 408

Notes:
Column 2 tested with round ends; All others with flat ends

○ – Elastic Limit.

● – Iron.

■ – Steel.

△ – Adjusted to 60 000 lb. Average ultimate x $\frac{60}{47}$

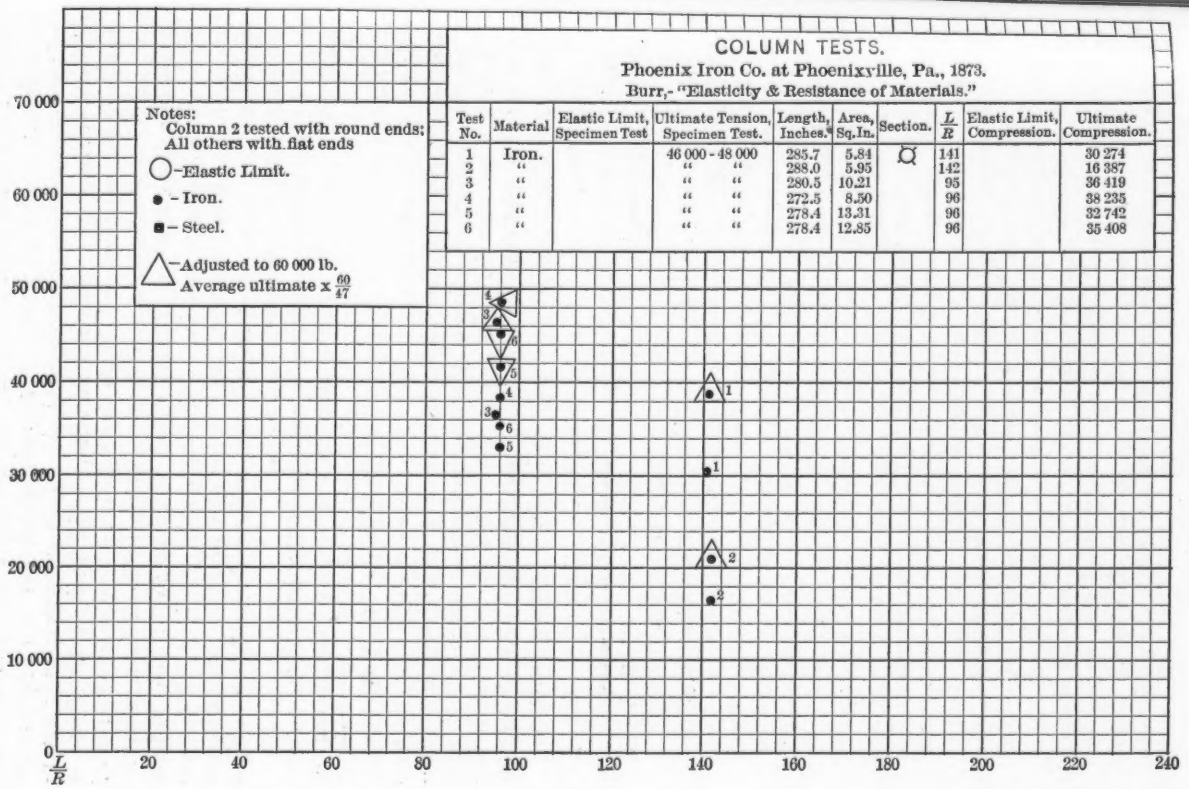


FIG. 1.

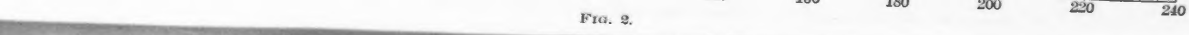


FIG. 2.

COLUMN TESTS.

G. Bouscaren.

Transactions, Am. Soc. C.E. Vol. IX, 1880.

Test No.	Material.	Elastic Limit, Specimen Test.	Ultimate Tension, Specimen Test.	Length, Inches.	Area, Sq. In.	Section.	$\frac{L}{R}$	Elastic Limit, Compression.	Ultimate Compression.
21	Iron.		52 000 - 55 000	309	13.600	][	93	18 000	25 500
22	"		" "	312	13.600	"	102	16 000	30 000
23	"		" "	288	13.700	"	84	15 000	33 200
32	"		" "	324	26.050	"	98	15 000	30 200
36	"		" "	408	7.480	[]	138	21 600	23 128
37	"		" "	329.7	12.078	][	74		29 600
38	"		" "	276.1	13.476	"	61	23 000	32 300
41	"		" "	330	13.740	"	72	20 800	32 400
42	"		" "	330	11.052	"	72	21 000	32 300

Notes:
Columns 21 & 36 tested with Pin Ends;
All others with Flat Ends.

○ — Elastic Limit.
● — Iron.
■ — Steel.
△ — Adjusted to 60 000 lb.
Average ultimate x $\frac{60\,000}{53\,500}$

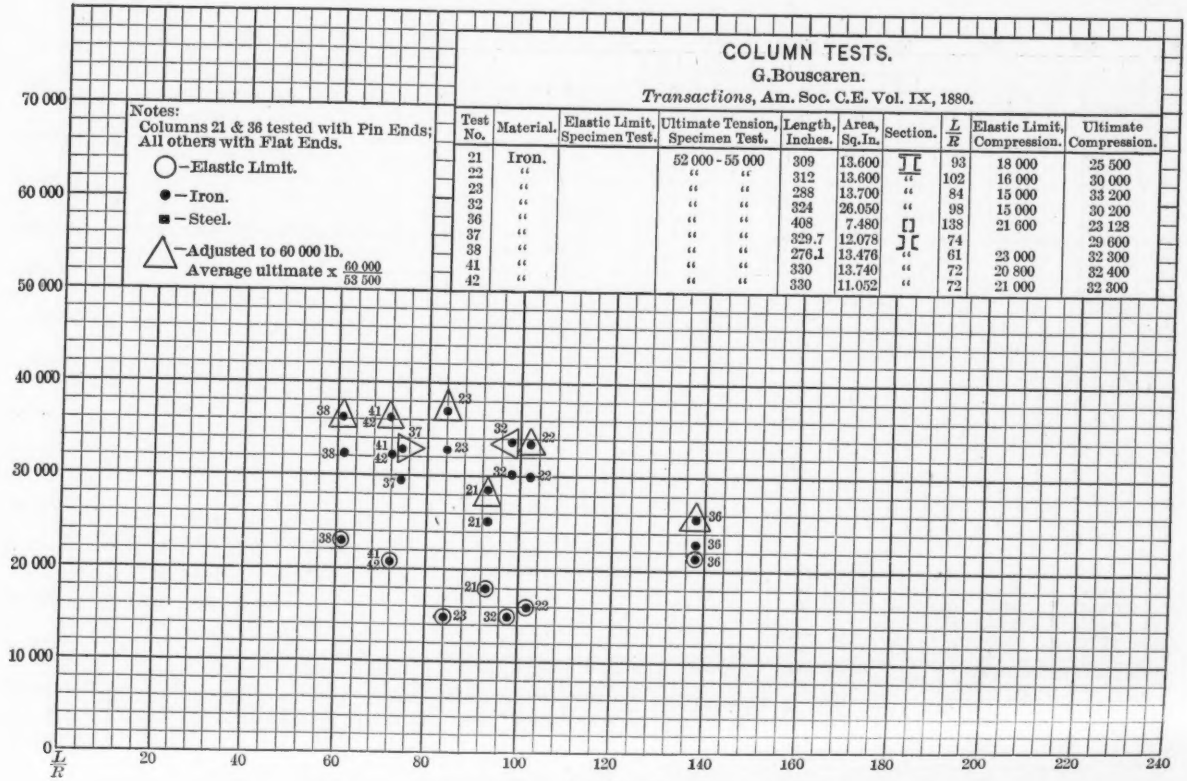


FIG. 2.

COLUMN TESTS.

Phoenix Iron Co. at Phoenixville, Pa., 1873.

COLUMN TESTS.

G. Bouscaren.

Transactions, Am. Soc. C.E. Vol. IX, 1880.

Test No.	Material.	Elastic Limit, Specimen Test.	Ultimate Tension, Specimen Test.	Length, Inches.	Area, Sq. In.	Section.	$\frac{L}{R}$	Elastic Limit, Compression.	Ultimate Compression.
6	Iron.		52 000–55 000	180	14 09	⌑	62	35 000	37 500
10	"		" "	324	13 70	"	108	18 000	31 000
11	"		" "	324	13 89	"	108	17 000	21 700
28	"		" "	336	13 58	"	112	22 000	34 800
29	"		" "	336	13 58	"	112	18 000	36 600

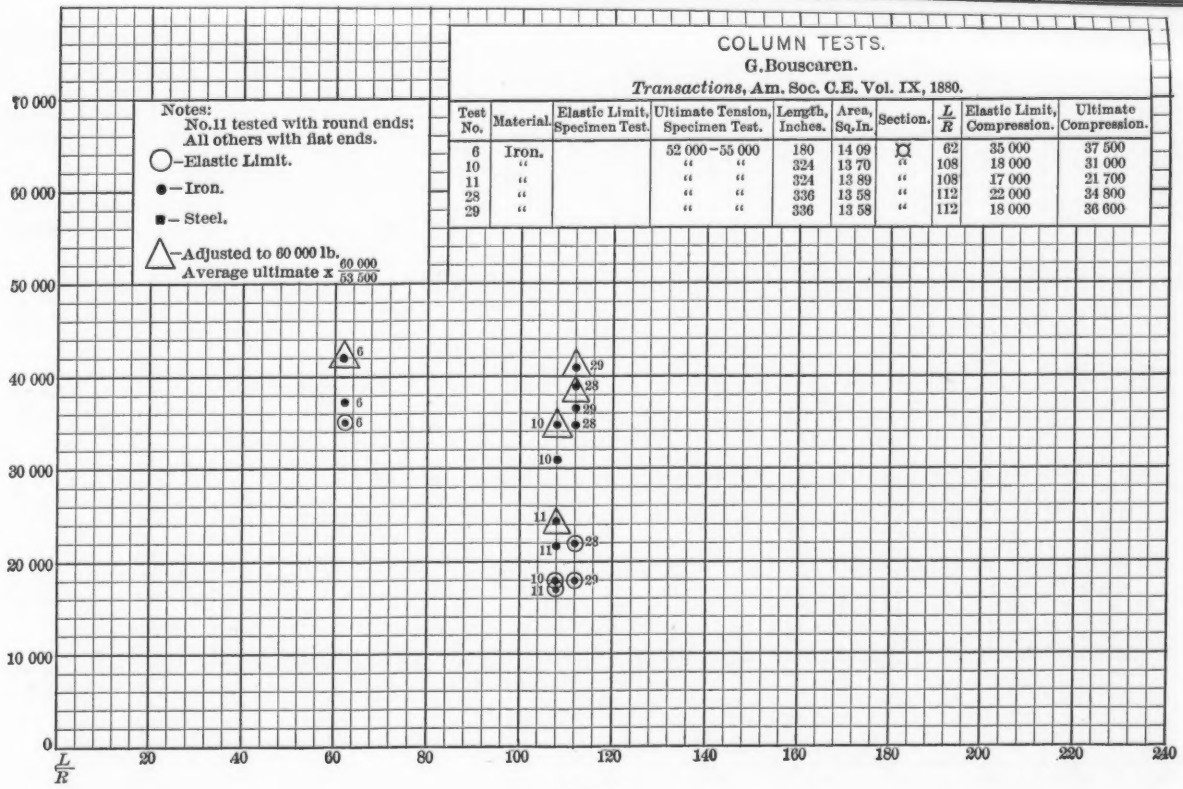
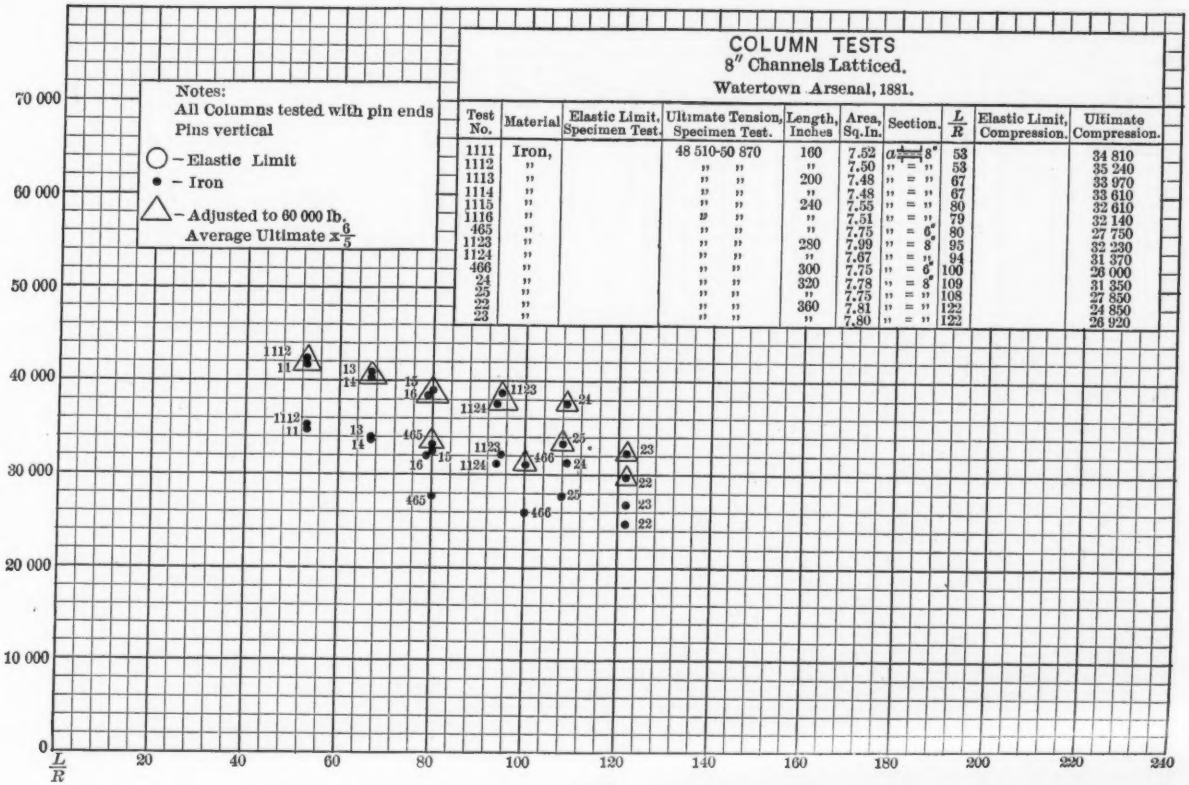


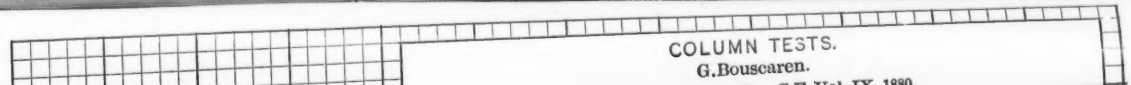
FIG. 3.

FIG. 4.



Test No.	Material	Elastic Limit, Specimen Test.	Ultimate Tension, Specimen Test.	Length, Inches	Area, Sq.In.	Section.	L/R	Elastic Limit, Compression.	Ultimate Compression.
1111	Iron,		48 510-50 870	160	7.52	$a\frac{1}{4}$ 8″	53		34 810
1112	,,		,, ,,	,,	7.50	,, = ,,	53		35 240
1113	,,		,, ,,	200	7.48	,, = ,,	67		33 970
1114	,,		,, ,,	,,	7.48	,, = ,,	67		33 610
1115	,,		,, ,,	240	7.55	,, = ,,	80		32 610
1116	,,		,, ,,	,,	7.51	,, = ,,	79		32 140
465	,,		,, ,,	,,	7.75	,, = 6″	80		27 750
1123	,,		,, ,,	280	7.99	,, = 8″	93		32 230
1124	,,		,, ,,	,,	7.67	,, = ,,	94		31 370
466	,,		,, ,,	300	7.75	,, = 6″	100		26 000
24	,,		,, ,,	320	7.78	,, = 8″	109		31 350
25	,,		,, ,,	,,	7.75	,, = ,,	108		27 850
22	,,		,, ,,	360	7.81	,, = ,,	122		24 850
23	,,		,, ,,	,,	7.80	,, = ,,	122		26 920

FIG. 4.



COLUMN TESTS.
10" Channels Latticed.
Watertown Arsenal, 1881.

Test No.	Material,	Elastic Limit, Specimen Test	Ultimate Tension, Specimen Test	Length, Inches	Area, Sq.In.	Section	$\frac{L}{R}$	Elastic Limit, Compression	Ultimate Compression
13	Iron		48 510-50 870	150	9.68	$a\frac{1}{4}$ $\frac{1}{4}$ $8''$	42		35 550
14	"		" "	"	9.59	" = "	42		35 350
11	"		" "	200	9.55	" = "	56		33 840
12	"		" "	"	9.61	" = "	56		34 000
467	"		" "	240	9.19	" = 6"	66		29 980
3	"		" "	250	9.74	" = 8"	70		33 880
4	"		" "	"	9.81	" = "	70		33 660
5	"		" "	300	10.04	" = "	85		31 130
6	"		" "	"	10.00	" = "	85		31 930
468	"		" "	"	9.19	" = 6"	82		32 000
26	"		" "	350	9.30	" = 8"	96		32 180
27	"		" "	"	9.57	" = "	97		29 380

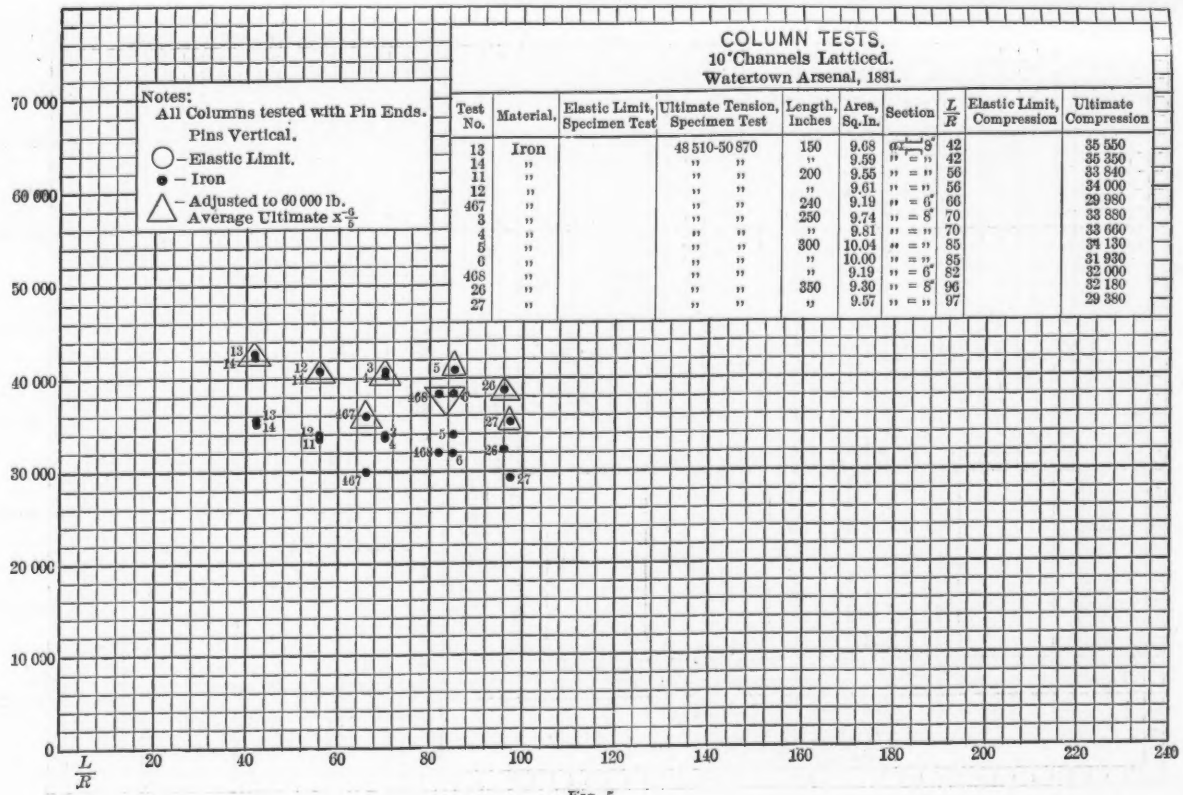
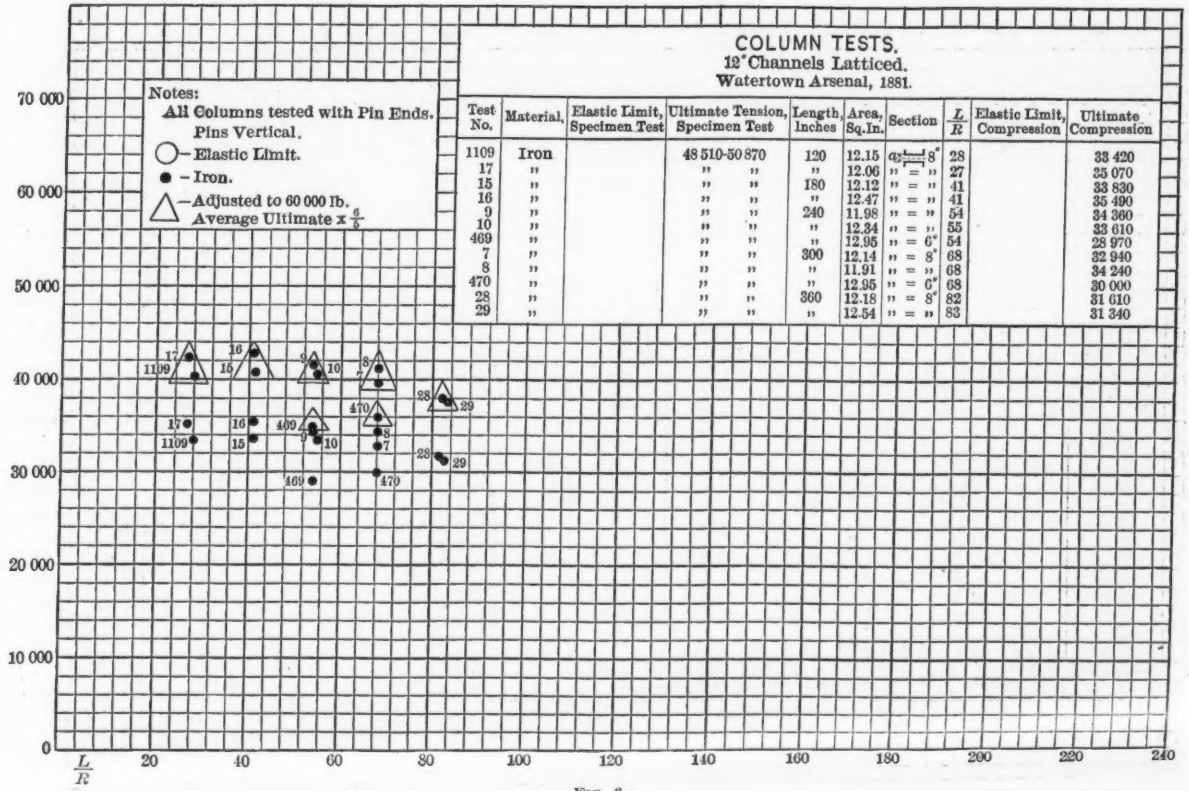


FIG. 5.

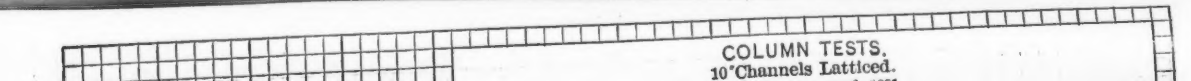
FIG. 6.

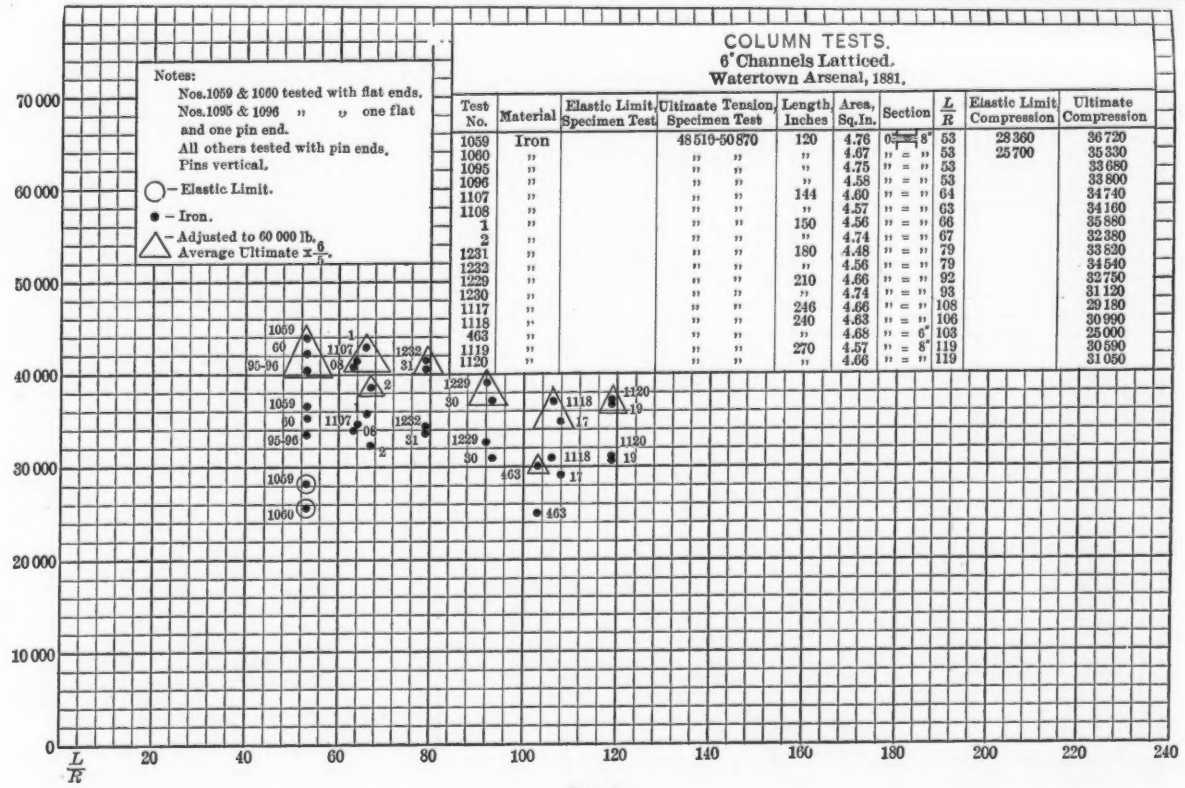


COLUMN TESTS.
12″ Channels Latticed.
Watertown Arsenal, 1881.

Test No.	Material.	Elastic Limit, Specimen Test	Ultimate Tension, Specimen Test	Length, Inches	Area, Sq.In.	Section	L/R	Elastic Limit, Compression	Ultimate Compression
1109	Iron		48 510-50 870	120	12.15	a 1/2 ⊏ 8″	28		33 420
17	,,		,, ,,	,,	12.06	,, = ,,	27		25 070
15	,,		,, ,,	180	12.12	,, = ,,	41		33 830
16	,,		,, ,,	,,	12.47	,, = ,,	41		35 490
9	,,		,, ,,	240	11.98	,, = ,,	54		34 360
10	,,		,, ,,	,,	12.34	,, = ,,	55		33 610
469	,,		,, ,,	,,	12.95	,, = 6″	54		28 970
7	,,		,, ,,	300	12.14	,, = 8″	68		32 940
8	,,		,, ,,	,,	11.91	,, = ,,	68		34 240
470	,,		,, ,,	,,	12.95	,, = 6″	68		30 000
28	,,		,, ,,	360	12.18	,, = 8″	82		31 610
29	,,		,, ,,	,,	12.54	,, = ,,	83		31 340

FIG. 6.

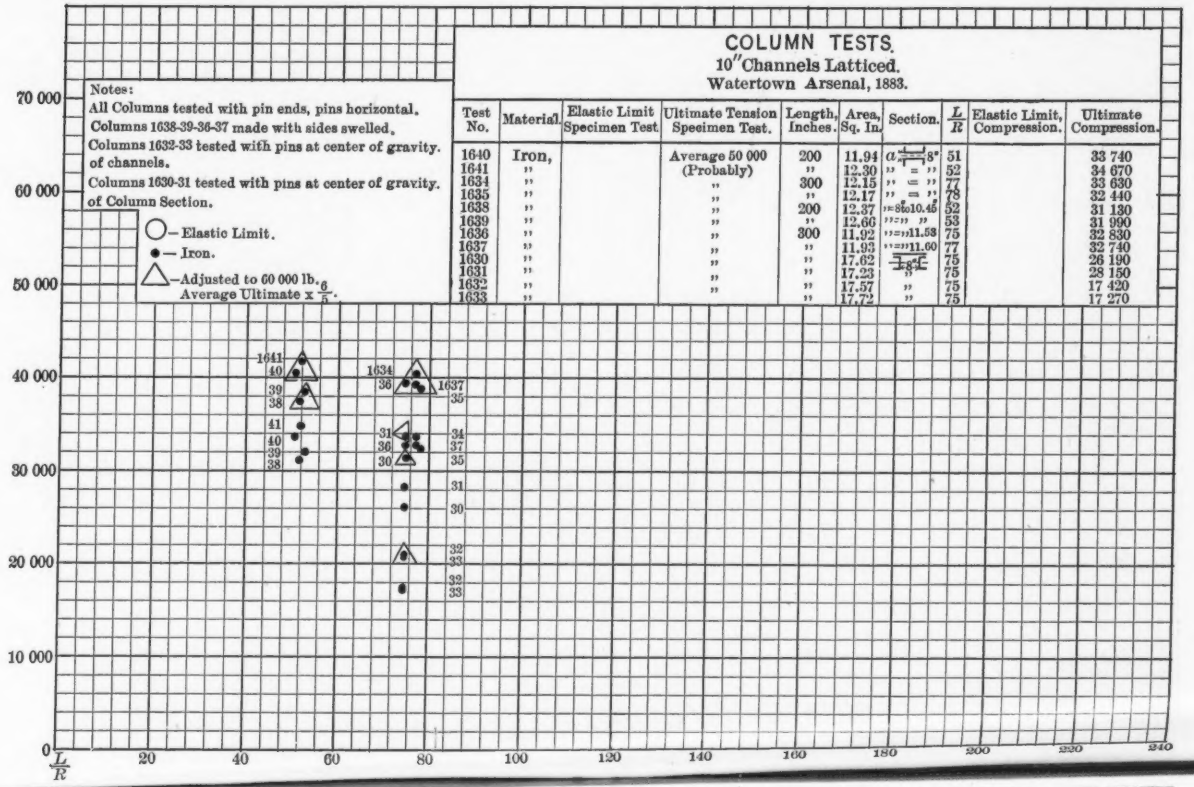




COLUMN TESTS.
6" Channels Latticed.
Watertown Arsenal, 1881.

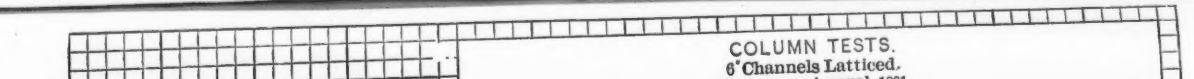
Test No.	Material	Elastic Limit, Specimen Test	Ultimate Tension, Specimen Test	Length, Inches	Area, Sq. In.	Section	L/R	Elastic Limit Compression	Ultimate Compression
1059	Iron		48 510-50 870	120	4.76	0 [8"	53	28 360	36 720
1060	"		" "	"	4.67	" = "	53	25 700	35 330
1095	"		" "	"	4.75	" = "	53		33 680
1096	"		" "	"	4.58	" = "	53		33 800
1107	"		" "	144	4.60	" = "	64		34 740
1108	"		" "	"	4.57	" = "	63		34 160
1	"		" "	150	4.56	" = "	66		35 880
2	"		" "	"	4.74	" = "	67		32 380
1231	"		" "	180	4.48	" = "	79		33 620
1232	"		" "	"	4.56	" = "	79		34 540
1229	"		" "	210	4.66	" = "	92		32 750
1230	"		" "	"	4.74	" = "	93		31 120
1117	"		" "	246	4.66	" = "	108		29 180
1118	"		" "	240	4.63	" = "	106		30 990
463	"		" "	"	4.68	" = 6"	103		25 000
1119	"		" "	270	4.57	" = 8"	119		30 590
1120	"		" "	"	4.66	" = "	119		31 050

FIG. 7.



COLUMN TESTS.
10″ Channels Latticed.
Watertown Arsenal, 1883.

Test No.	Material	Elastic Limit Specimen Test.	Ultimate Tension Specimen Test.	Length, Inches.	Area, Sq. In.	Section.	L/R	Elastic Limit, Compression.	Ultimate Compression.
1640	Iron,		Average 50 000	200	11.94	α ⊏ 8″	51		33 740
1641	,,		(Probably)	,,	12.30	,, = ,,	52		34 670
1634	,,		,,	300	12.15	,, ⊏ ,,	77		33 630
1635	,,		,,	,,	12.17	,, ⊏ ,,	78		32 440
1638	,,		,,	200	12.37	⊏ 8 to 10.45	52		31 130
1639	,,		,,	,,	12.66	,, = ,,	53		31 990
1636	,,		,,	300	11.92	,, = 11.53	75		32 830
1637	,,		,,	,,	11.93	,, = 11.60	77		32 740
1630	,,		,,	,,	17.62	⊏ 8 to 8	75		26 190
1631	,,		,,	,,	17.23	,,	75		28 150
1632	,,		,,	,,	17.57	,,	75		17 420
1633	,,		,,	,,	17.72	,,	75		17 270



COLUMN TESTS.
6″ Channels Latticed.

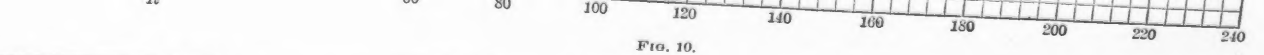
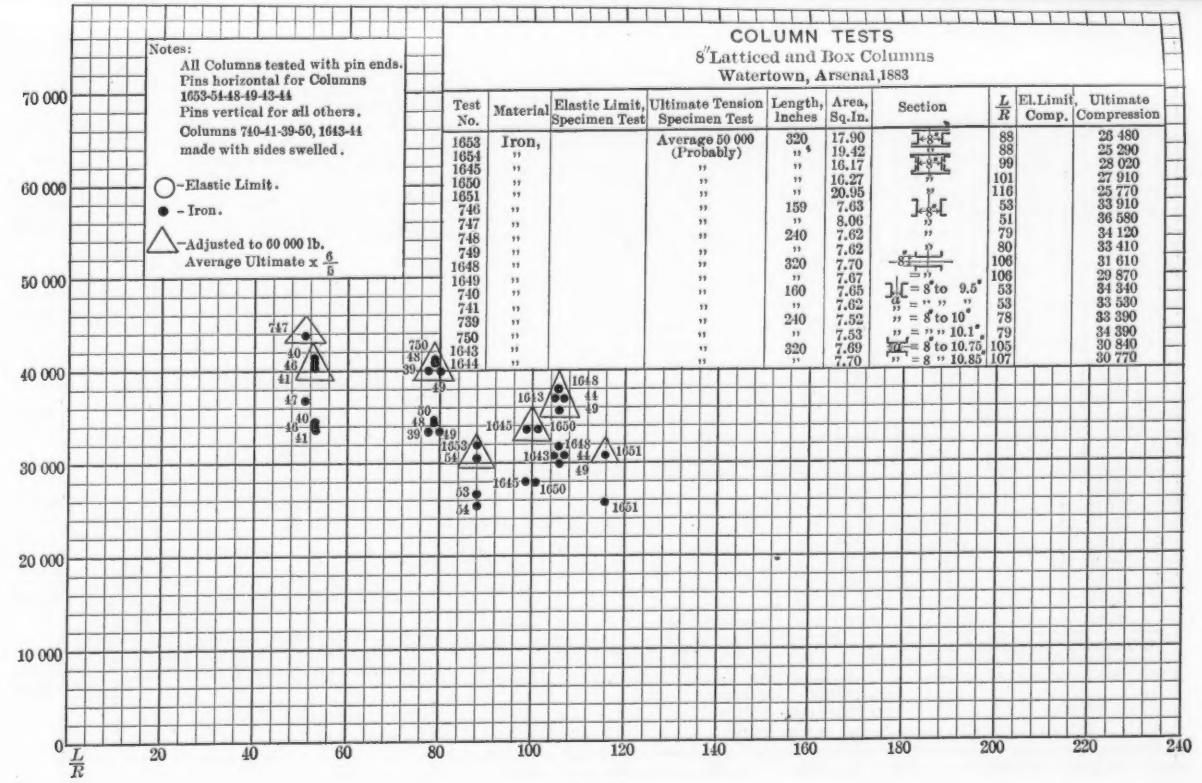


FIG. 10.



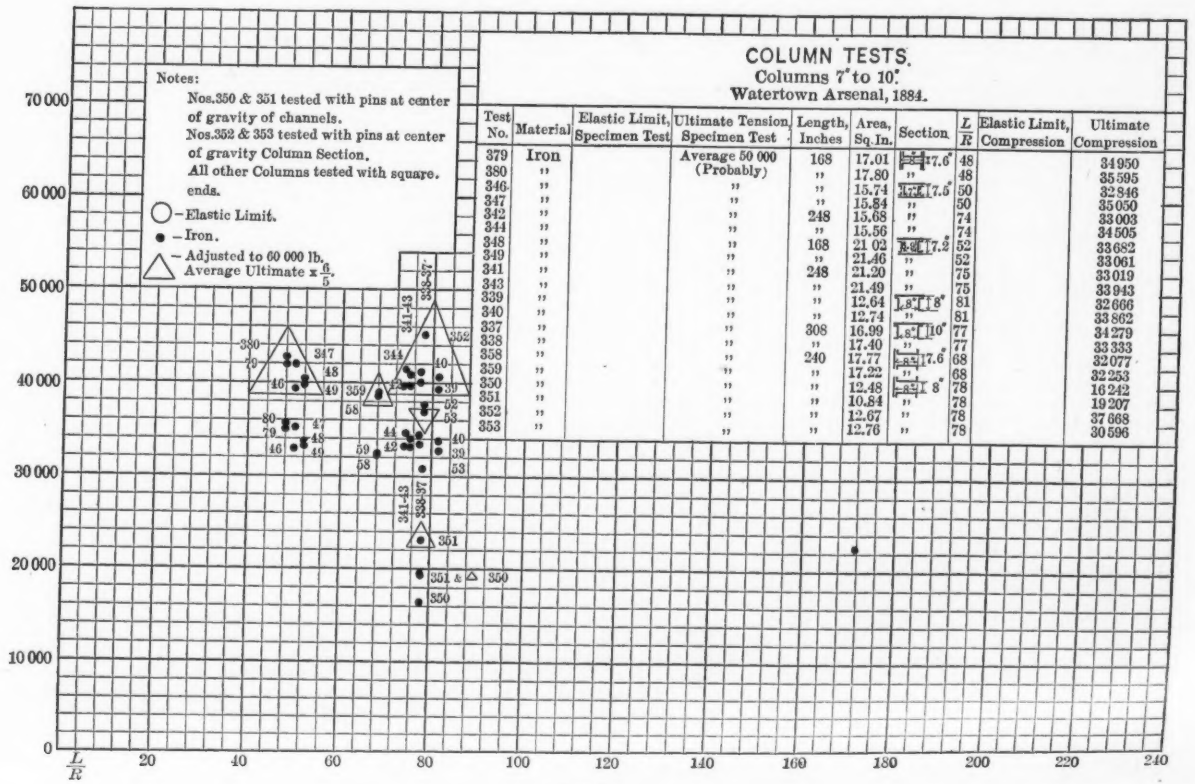
COLUMN TESTS

8″ Latticed and Box Columns

Watertown, Arsenal, 1883

Test No.	Material	Elastic Limit, Specimen Test	Ultimate Tension Specimen Test	Length, Inches	Area, Sq. In.	Section	$\frac{L}{R}$	El. Limit, Comp.	Ultimate Compression
1653	Iron,		Average 50 000 (Probably)	320	17.90	[8″×8″	88		26 480
1654	”		”	”	19.42	[8″×8″	88		25 290
1645	”		”	”	16.17	[8″×8″	99		28 020
1650	”		”	”	16.27	”	101		27 910
1651	”		”	”	20.95	”	116		25 770
746	”		”	159	7.63	[8″	53		33 910
747	”		”	”	8.06	”	51		36 580
748	”		”	240	7.62	”	79		34 120
749	”		”	”	7.62	”	80		33 410
1648	”		”	320	7.70	[8″	106		31 610
1649	”		”	”	7.67	= ”	106		29 870
740	”		”	160	7.65	[= 8″ to 9.5″	53		34 340
741	”		”	”	7.62	” = ” ” ”	53		33 530
739	”		”	240	7.52	” = 8″ to 10″	78		33 390
750	”		”	”	7.53	” = ” ” 10.1″	79		34 390
1643	”		”	320	7.69	[= 8″ to 10.75″	105		30 840
1644	”		”	”	7.70	” = 8″ ” 10.85″	107		30 770

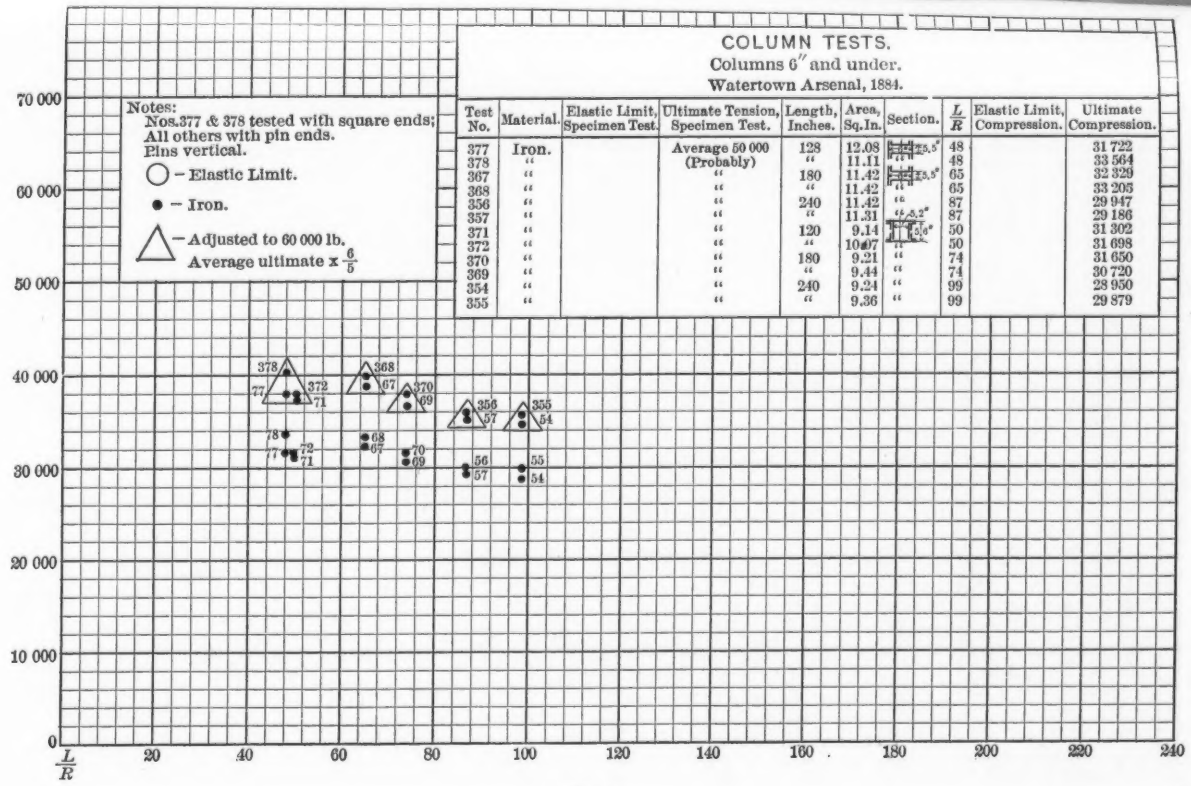
FIG. 9.



COLUMN TESTS.
Columns 7" to 10".
Watertown Arsenal, 1884.

Test No.	Material	Elastic Limit, Specimen Test	Ultimate Tension, Specimen Test	Length, Inches	Area, Sq. In.	Section	L/R	Elastic Limit, Compression	Ultimate Compression
379	Iron		Average 50 000 (Probably)	168	17.01	7.6"	48		34950
380	,,		,,	,,	17.80	,,	48		35595
346	,,		,,	,,	15.74	7.5"	50		32846
347	,,		,,	,,	15.84	,,	50		35050
342	,,		,,	248	15.68	,,	74		33003
344	,,		,,	,,	15.56	,,	74		34505
348	,,		,,	168	21.02	7.2"	52		33682
349	,,		,,	,,	21.46	,,	52		33061
341	,,		,,	248	21.20	,,	75		33019
343	,,		,,	,,	21.49	,,	75		33943
339	,,		,,	,,	12.64	8"	81		32666
340	,,		,,	,,	12.74	,,	81		33862
337	,,		,,	308	16.99	10"	77		34279
338	,,		,,	,,	17.40	,,	77		33333
358	,,		,,	240	17.77	7.6"	68		32077
359	,,		,,	,,	17.22	,,	68		32253
350	,,		,,	,,	12.48	8"	78		16242
351	,,		,,	,,	10.84	,,	78		19207
352	,,		,,	,,	12.67	,,	78		37668
353	,,		,,	,,	12.76	,,	78		30596

FIG. 10.



COLUMN TESTS.

Columns 6″ and under.

Watertown Arsenal, 1884.

Test No.	Material.	Elastic Limit, Specimen Test.	Ultimate Tension, Specimen Test.	Length, Inches.	Area, Sq.In.	Section.	$\frac{L}{R}$	Elastic Limit, Compression.	Ultimate Compression.
377	Iron.		Average 50 000 (Probably)	128	12.08	5.5″	48		31 722
378	"		"	"	11.11	5.5″	48		33 564
367	"		"	180	11.42	5.5″	65		32 329
368	"		"	"	11.42	"	65		33 203
356	"		"	240	11.42	"	87		29 947
357	"		"	"	11.31	4.2″	87		29 186
371	"		"	120	9.14	3.6″	50		31 302
372	"		"	"	10.07	"	50		31 698
370	"		"	180	9.21	"	74		31 650
369	"		"	"	9.44	"	74		30 720
354	"		"	240	9.24	"	99		28 950
355	"		"	"	9.36	"	99		29 879

FIG. 11.

FIG. 12.

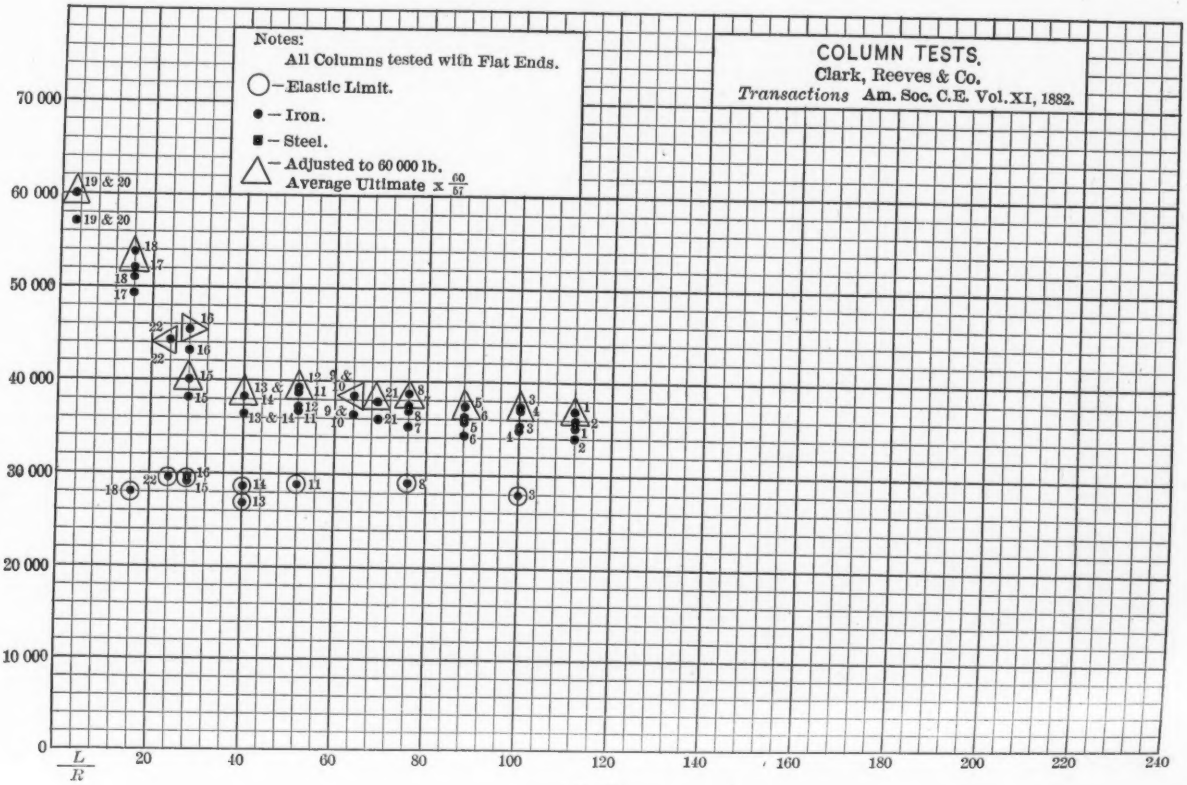
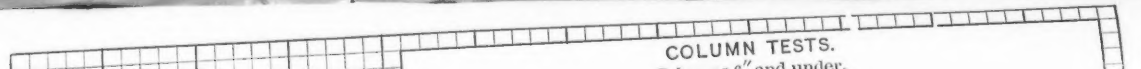

Notes:
All Columns tested with Flat Ends.
○ — Elastic Limit.
● — Iron.
■ — Steel.
△ — Adjusted to 60 000 lb.
Average Ultimate x 60/67
COLUMN TESTS.
Clark, Reeves & Co.
Transactions Am. Soc. C.E. Vol. XI, 1882.
70 000
60 000
50 000
40 000
30 000
20 000
10 000
0
L/R
20
40
60
80
100
120
140
160
180
200
220
240

FIG. 12.


COLUMN TESTS.

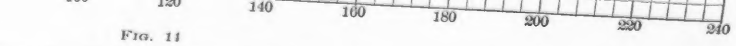
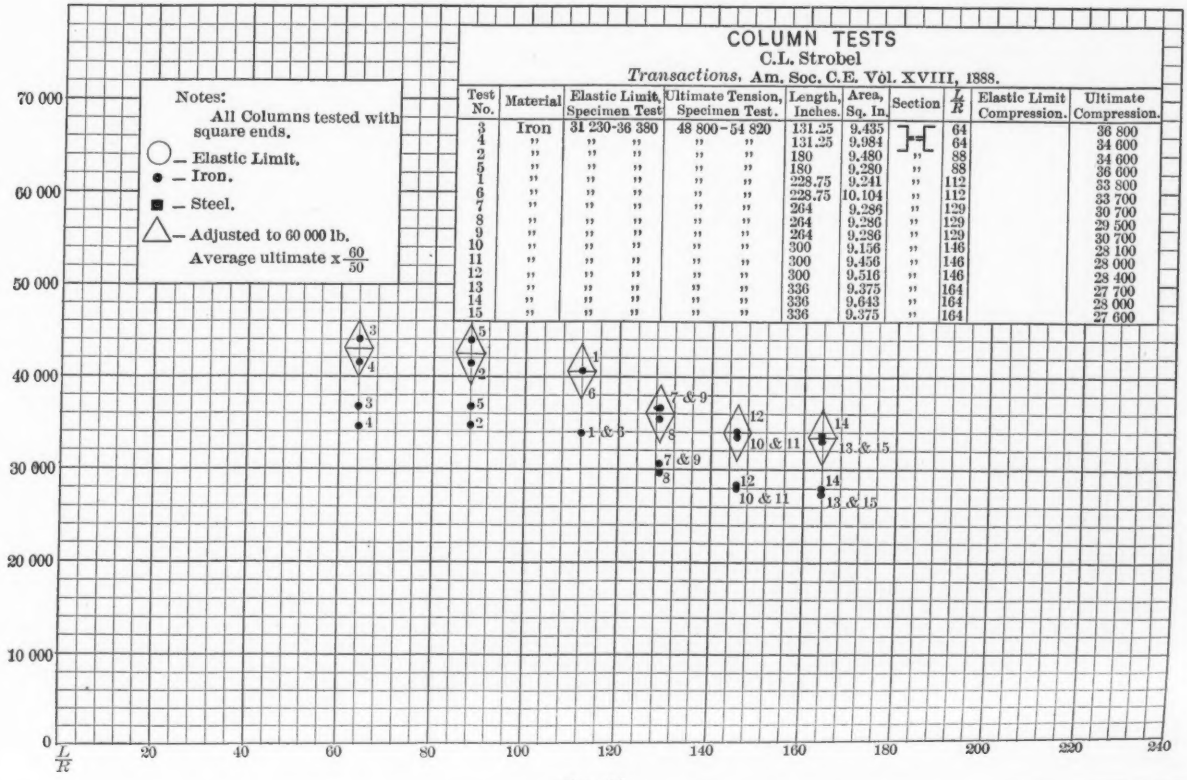


FIG. 11

COLUMN TESTS
Clark, Reeves & Co.
Transactions, Am. Soc. C.E. Vol. XI, 1882.

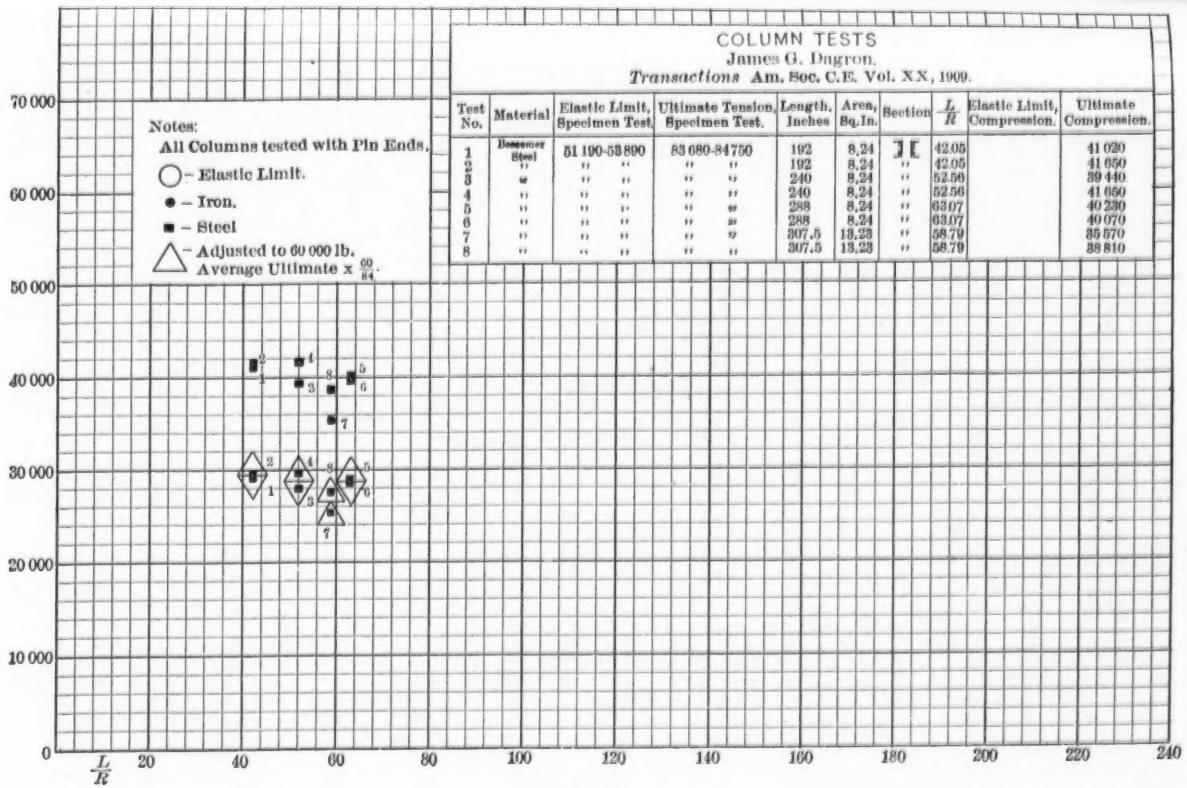
Test No.	Material	Elastic Limit, Specimen Test.	Ultimate Tension, Specimen Test.	Length, Inches.	Area, Sq. In.	Section.	$\frac{L}{R}$	Elastic Limit, Compression	Ultimate Compression.
1	Iron		Clark, Reeves & Co's Specification of 1871 Calls for 55 000 to 60 000	336	12.062	▯	112		35 150
2	"			336	12.181	"	112		34 150
3	"			300	12.233	"	100	27 960	35 270
4	"			300	12.100	"	100		35 040
5	"			264	12.371	"	88		35 570
6	"			264	12.311	"	88		34 360
7	"			228	12.023	"	76		35 365
8	"			228	12.087	"	76	29 290	36 900
9	"			192	12.000	"	64		36 380
10	"			192	12.000	"	64		36 580
11	"			156	12.185	"	52	28 890	36 857
12	"			156	12.069	"	52		37 200
13	"			120	12.248	"	40	26 940	36 480
14	"			120	12.339	"	40	28 360	36 397
15	"			84	12.265	"	28	29 350	38 157
16	"			84	11.962	"	28	29 590	43 300
17	"			48	12.081	"	16		49 500
18	"			48	12.119	"	16	28 050	51 240
19	"			8	11.903	"	2.7		57 130
20	"			8	11.903	"	2.7		57 300
21	"			302.65	18.300	☆	68.8		36 010
22	"			105.50	18.300	"	24	29 510	42 180

FIG. 13.



Test No.	Material	Elastic Limit, Specimen Test	Ultimate Tension, Specimen Test.	Length, Inches.	Area, Sq. In.	Section	L/r	Elastic Limit Compression.	Ultimate Compression.
3	Iron	31 230-36 380	48 800-54 820	131.25	9.435	⊢⊣	64		36 800
4	”	” ”	” ”	131.25	9.984	”	64		34 600
2	”	” ”	” ”	180	9.480	”	88		34 600
5	”	” ”	” ”	180	9.280	”	88		36 600
1	”	” ”	” ”	228.75	9.241	”	112		33 800
6	”	” ”	” ”	228.75	10.104	”	112		33 700
7	”	” ”	” ”	264	9.286	”	129		30 700
8	”	” ”	” ”	264	9.286	”	129		29 500
9	”	” ”	” ”	264	9.286	”	129		30 700
10	”	” ”	” ”	300	9.156	”	146		28 100
11	”	” ”	” ”	300	9.456	”	146		28 000
12	”	” ”	” ”	300	9.516	”	146		28 400
13	”	” ”	” ”	336	9.375	”	164		27 700
14	”	” ”	” ”	336	9.643	”	164		28 000
15	”	” ”	” ”	336	9.375	”	164		27 600

FIG. 14



Test No.	Material	Elastic Limit, Specimen Test.	Ultimate Tension, Specimen Test.	Length, Inches	Area, Sq. In.	Section	$\frac{L}{R}$	Elastic Limit, Compression.	Ultimate Compression.
1	Bessemer Steel	51 190-53 890	83 680-84 750	192	8.24	][	42.05		41 020
2	"	" "	" "	192	8.24	"	42.05		41 050
3	"	" "	" "	240	8.24	"	52.56		39 440
4	"	" "	" "	240	8.24	"	52.56		41 050
5	"	" "	" "	288	8.24	"	63.07		40 230
6	"	" "	" "	288	8.24	"	63.07		40 070
7	"	" "	" "	307.5	13.23	"	58.79		35 570
8	"	" "	" "	307.5	13.23	"	58.79		38 810

FIG. 15

COLUMN TESTS

C.P. Buchanan.

Engineering News Vol. 58, Dec. '07, Page 685.

Test No.	Material	Elastic Limit, Specimen Test	Ultimate Tension, Specimen Test.	Length, Inches	Area, Sq. In.	Section	$\frac{L}{R}$	Elastic Limit, Compression.	Ultimate Compression.
1	Iron			300	17.78	H	98		24 800
2	"			300	17.56	"	120		26 350
3	Steel		Assumed	156	15.25	"	83		34 270
4	"		64 000	156	14.15	"	83		32 900
9	"	39 000	62 000-65 000	300	14.22	[ ]	97	12 600	27 790
13	Iron	30 070-31 000	48 440-51 910	242	27.28	][	42	20 150	28 200
14	"	29 650-31 440	50 000-52 630	190	21.78	][	40	17 200	26 260
15	"	30 230-32 250	49 100-51 240	249	30.81	][	37	17 000	30 300
16	Steel		63 000-66 000	297	29.36	"	46	25 500	30 640
17	"		63 000-68 000	168	15.80	][	45	19 000	31 680
18	Steel		61 000-65 000	249	32.11	][	34	22 600	31 080
19	"		61 000-65 000	249	32.16	"	35	18 700	33 950

Notes:

All Columns tested with Pin Ends.

○ – Elastic Limit.

● – Iron Columns.

■ – Steel "

△ – Adjusted to 60 000 lb.

Average Ultimate x $\frac{6}{5}$ for Iron

" " x $\frac{60}{64}$ " Steel

70 000
60 000
50 000
40 000
30 000
20 000
10 000
0

$\frac{L}{R}$ 20 40 60 80 100 120 140 160 180 200 220 240

FIG 16.

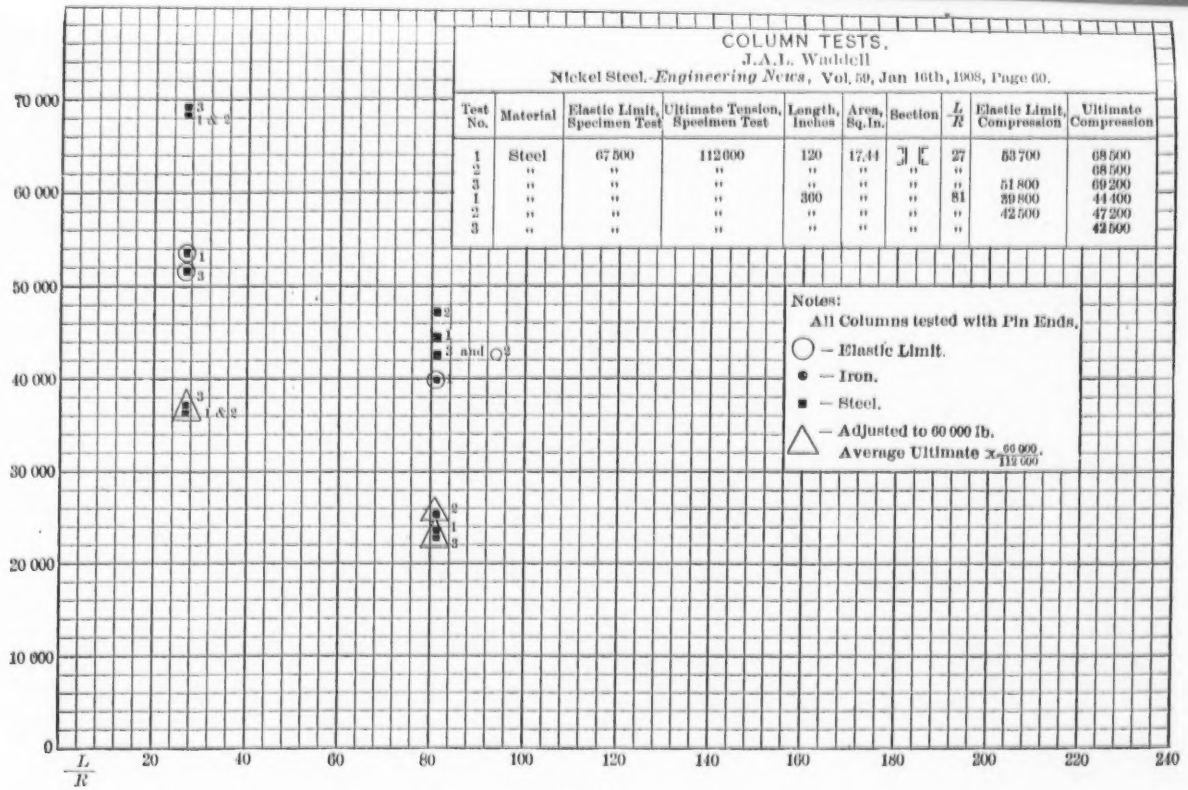
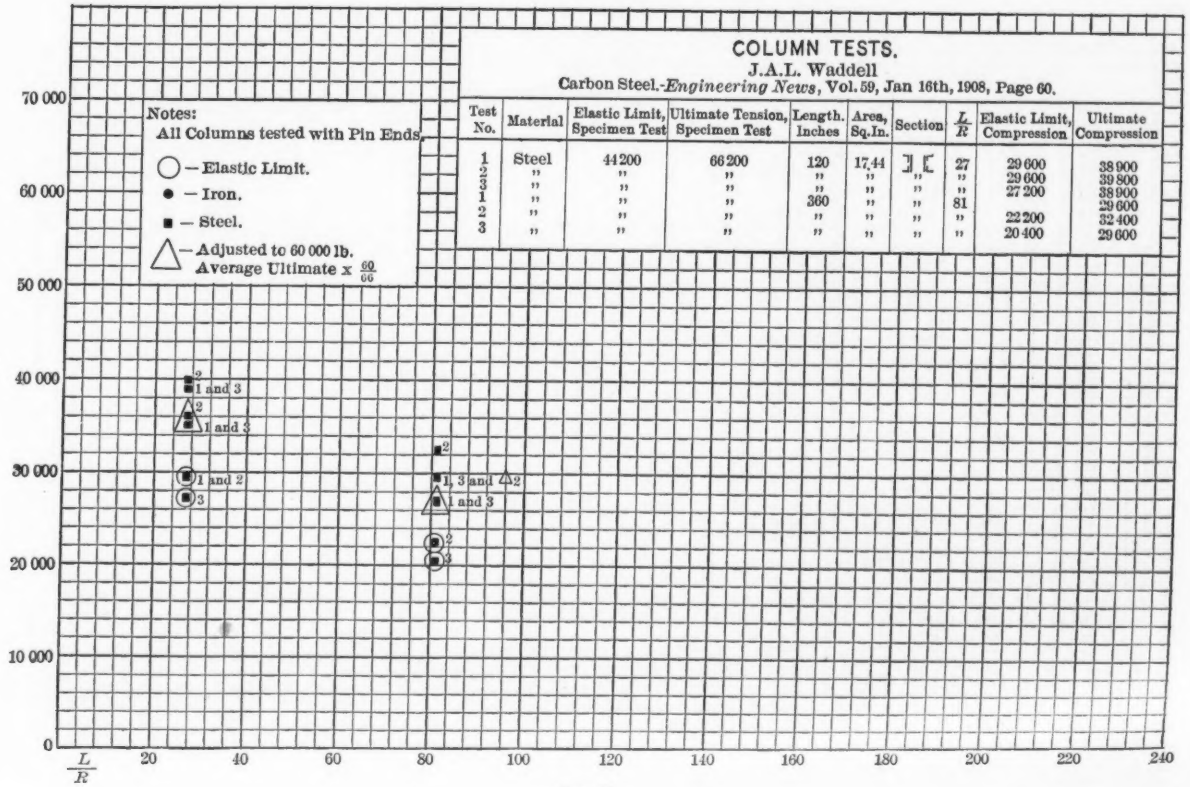

COLUMN TESTS.
J.A.L. Waddell
Nickel Steel.—Engineering News, Vol. 59, Jan 16th, 1908, Page 60.
Test No. | Material | Elastic Limit, Specimen Test | Ultimate Tension, Specimen Test | Length, Inches | Area, Sq. In. | Section | L/R | Elastic Limit, Compression | Ultimate Compression
1 | Steel | 67 500 | 112 000 | 120 | 17.44 |] [| 27 | 53 700 | 68 500
2 | " | " | " | " | " | " | " | | 68 500
3 | " | " | " | " | " | " | " | 51 800 | 69 200
1 | " | " | " | 300 | " | " | 81 | 39 800 | 44 400
2 | " | " | " | " | " | " | " | 42 500 | 47 200
3 | " | " | " | " | " | " | " | | 42 500
Notes:
All Columns tested with Pin Ends.
— Elastic Limit.
— Iron.
— Steel.
— Adjusted to 60 000 lb.
Average Ultimate $x \frac{60\,000}{112\,000}$.
70 000
60 000
50 000
40 000
30 000
20 000
10 000
0
L/R
20
40
60
80
100
120
140
160
180
200
220
240

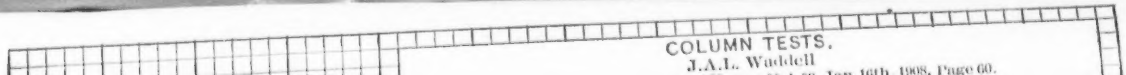
FIG. 17.

FIG. 18.



Test No.	Material	Elastic Limit, Specimen Test	Ultimate Tension, Specimen Test	Length. Inches	Area, Sq.In.	Section	L/R	Elastic Limit, Compression	Ultimate Compression
1	Steel	44200	66200	120	17.44	][	27	29600	38900
2	"	"	"	"	"	"	"	29600	39800
3	"	"	"	"	"	"	"	27200	38900
1	"	"	"	360	"	"	81		29600
2	"	"	"	"	"	"	"	22200	32400
3	"	"	"	"	"	"	"	20400	29600

FIG. 18.



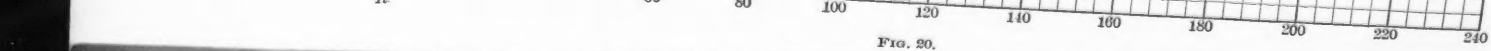


FIG. 20.

COLUMN TESTS.

Lap-Welded Steel Tubing.

Watertown Arsenal, "Tests of Metals," 1908.

Test No.	Material	Elastic Limit, Specimen Test	Ultimate Tension, Specimen Test	Length, Inches	Area, Sq. In.	Section	$\frac{L}{R}$	Elastic Limit, Compression	Ultimate Compression
1895	Steel	35 000-38 610	56 390-60 830	20.56	5.15	○	12.5	32 000	97 000
6	"	" "	" "	"	5.24	"	12.5	33 000	66 350
7	"	" "	" "	"	5.24	"	12.5	33 000	66 180
1891	"	" "	" "	41.00	5.34	"	25.0		41 700
2	"	" "	" "	"	5.25	"	"	30 000	40 610
3	"	" "	" "	"	5.17	"	"	29 000	39 850
1898	"	" "	" "	41.12	5.10	"	"	31 000	46 000
9	"	" "	" "	"	5.18	"	"	31 000	43 200
1900	"	" "	" "	"	5.36	"	"	29 000	41 420
1	"	" "	" "	61.69	5.06	"	37.5	32 000	34 000
2	"	" "	" "	"	5.23	"	"	30 000	32 000
3	"	" "	" "	"	5.34	"	"	27 000	31 740
1888	"	" "	" "	82.25	5.24	"	50		28 820
9	"	" "	" "	"	5.19	"	"		32 000
1890	"	" "	" "	"	5.21	"	"		32 000
1904	"	" Average: 36 250	" Average: 58 310	"	5.33	"	"	28 000	30 000
5	"	" "	" "	"	5.06	"	"	31 000	32 000
6	"	" "	" "	"	5.25	"	"	30 000	32 000
7	"	" "	" "	102.81	5.20	"	62.5	32 000	35 630
8	"	" "	" "	"	5.08	"	"		31 000
9	"	" "	" "	"	5.26	"	"	29 000	30 000
1894	"	" "	" "	120.50	5.16	"	73.3		33 000
1885	"	" "	" "	123.50	5.23	"	75		33 000
6	"	" "	" "	"	5.19	"	"		33 000
7	"	" "	" "	"	5.20	"	"		31 000
1882	"	" "	" "	164.50	5.26	"	100		31 000
3	"	" "	" "	"	5.21	"	"		28 000
4	"	" "	" "	"	5.17	"	"		31 000
1879	"	" "	" "	205.50	5.21	"	125		29 000
80	"	" "	" "	"	5.07	"	"		26 030
1	"	" "	" "	"	5.21	"	"		29 000
1876	"	" "	" "	246.75	5.11	"	150		26 000
7	"	" "	" "	"	5.17	"	"		27 000
8	"	" "	" "	"	5.16	"	"		23 000
1873	"	" "	" "	288.00	5.04	"	175		25 000
4	"	" "	" "	"	5.06	"	"		24 000
5	"	" "	" "	"	5.20	"	"		27 000

FIG. 19.

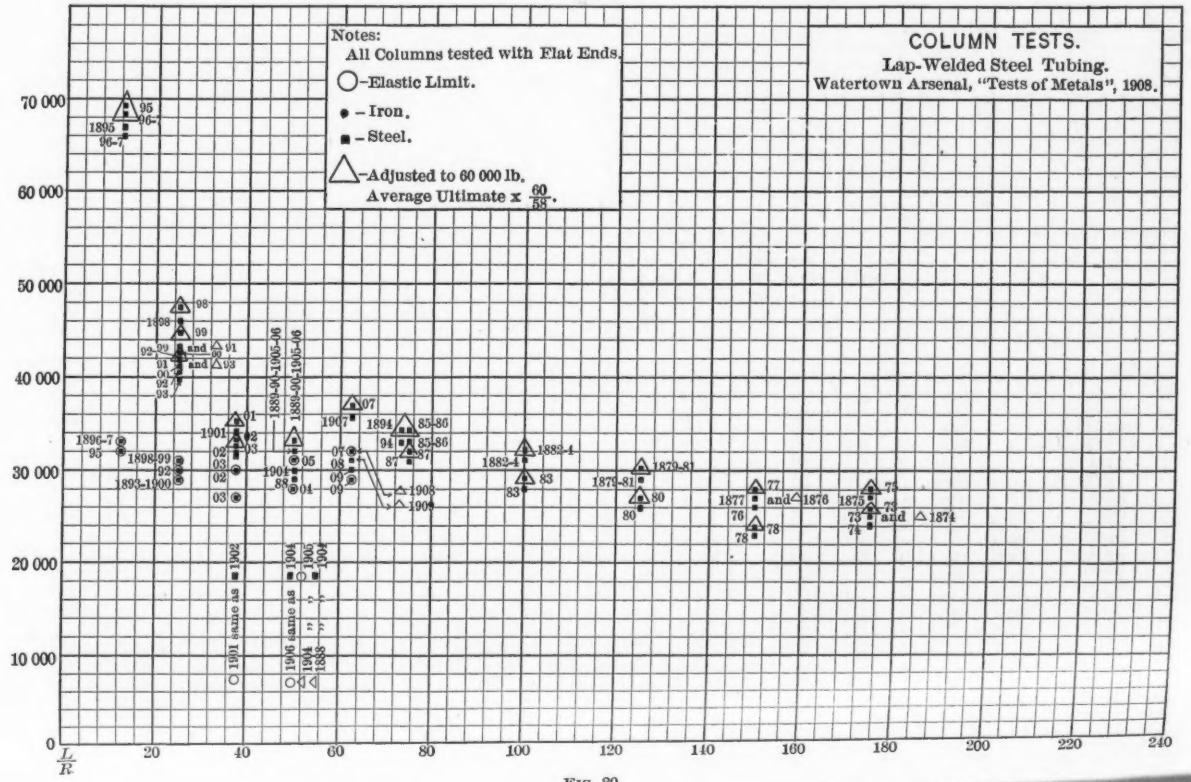
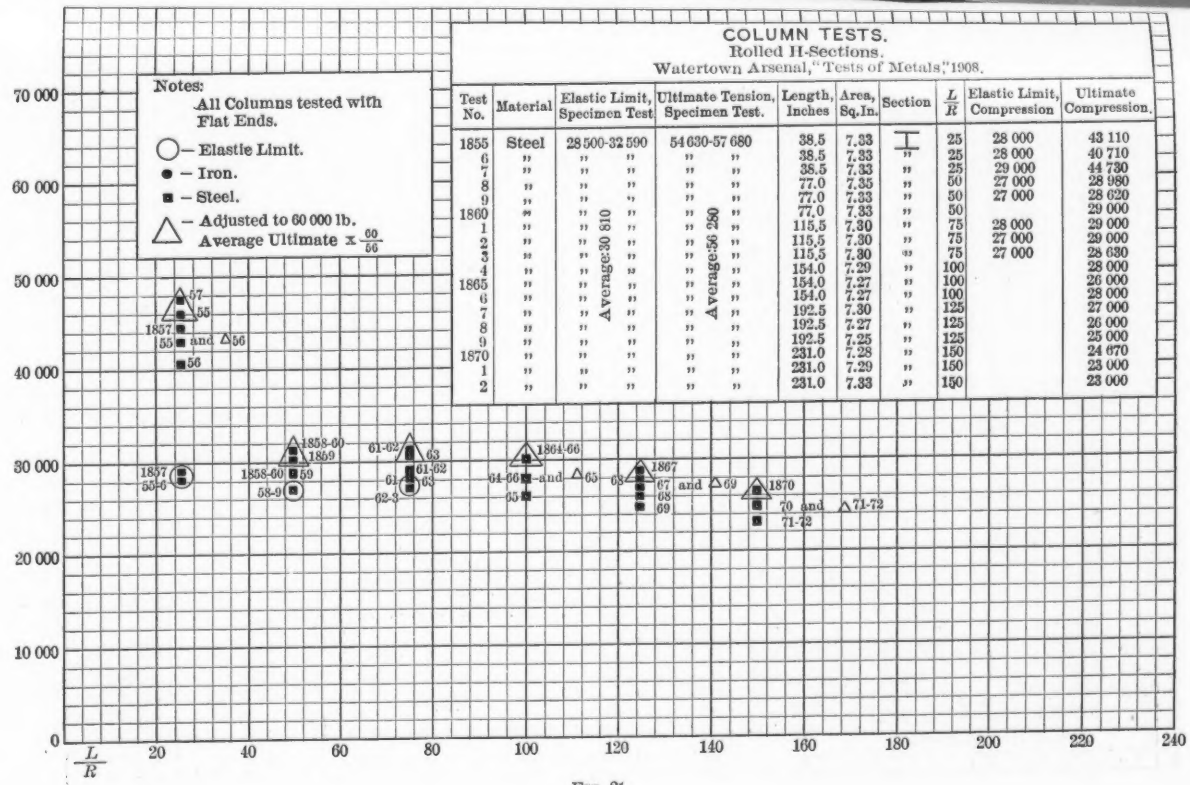

COLUMN TESTS.
Lap-Welded Steel Tubing.
Watertown Arsenal, "Tests of Metals", 1908.
Notes:
All Columns tested with Flat Ends.
○–Elastic Limit.
• – Iron.
■ – Steel.
△–Adjusted to 60 000 lb.
Average Ultimate x 60/55.
70 000
60 000
50 000
40 000
30 000
20 000
10 000
0
L/R
20
40
60
80
100
120
140
160
180
200
220
240

FIG. 20.



COLUMN TESTS.
Rolled H-Sections.
Watertown Arsenal, "Tests of Metals," 1908.

Test No.	Material	Elastic Limit, Specimen Test	Ultimate Tension, Specimen Test.	Length, Inches	Area, Sq. In.	Section	$\frac{L}{R}$	Elastic Limit, Compression	Ultimate Compression.
1855	Steel	28 500-32 590	54 630-57 680	38.5	7.33	I	25	28 000	43 110
6	"	" "	" "	38.5	7.33	"	25	28 000	40 710
7	"	" "	" "	38.5	7.33	"	25	29 000	44 730
8	"	" "	" "	77.0	7.35	"	50	27 000	28 980
9	"	" "	" "	77.0	7.33	"	50	27 000	28 620
1860	"	" "	" "	77.0	7.33	"	50		29 000
1	"	" "	" "	115.5	7.30	"	75	28 000	29 000
2	"	" "	" "	115.5	7.30	"	75	27 000	29 000
3	"	" "	" "	115.5	7.30	"	75	27 000	28 630
4	"	" "	" "	154.0	7.29	"	100		28 000
1865	"	" "	" "	154.0	7.27	"	100		26 000
6	"	" "	" "	154.0	7.27	"	100		28 000
7	"	" "	" "	192.5	7.30	"	125		27 000
8	"	" "	" "	192.5	7.27	"	125		26 000
9	"	" "	" "	192.5	7.25	"	125		25 000
1870	"	" "	" "	231.0	7.28	"	150		24 670
1	"	" "	" "	231.0	7.29	"	150		23 000
2	"	" "	" "	231.0	7.33	"	150		23 000

Average: 30 810 — Average: 56 260

FIG. 21.

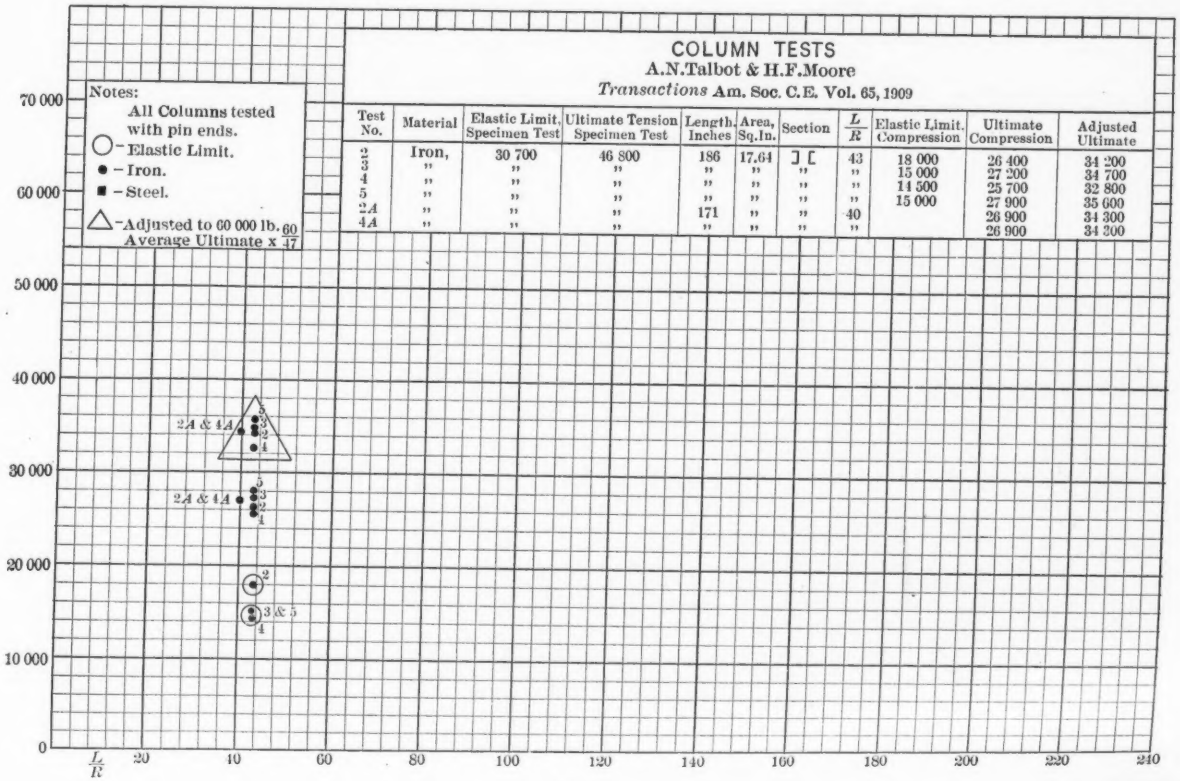
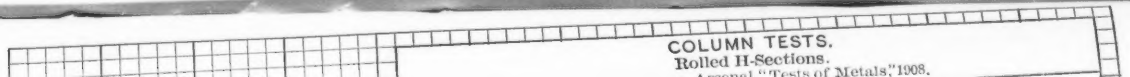

COLUMN TESTS
A.N.Talbot & H.F.Moore
Transactions Am. Soc. C.E. Vol. 65, 1909
Notes:
All Columns tested with pin ends.
○ – Elastic Limit.
● – Iron.
■ – Steel.
△ – Adjusted to 60 000 lb. 60/47 Average Ultimate x
Test No. | Material | Elastic Limit, Specimen Test | Ultimate Tension Specimen Test | Length, Inches | Area, Sq.In. | Section | L/R | Elastic Limit, Compression | Ultimate Compression | Adjusted Ultimate
2 | Iron, | 30 700 | 46 800 | 186 | 17.64 |] [| 43 | 18 000 | 26 400 | 34 200
3 | " | " | " | " | " | " | " | 15 000 | 27 200 | 34 700
4 | " | " | " | " | " | " | " | 14 500 | 25 700 | 32 800
5 | " | " | " | " | " | " | " | 15 000 | 27 900 | 35 600
2A | " | " | " | 171 | " | " | 40 | | 26 900 | 34 300
4A | " | " | " | " | " | " | " | | 26 900 | 34 300
70 000
60 000
50 000
40 000
30 000
20 000
10 000
0
L/R
20
40
60
80
100
120
140
160
180
200
220
240
2A & 4A
2A & 4A
3 & 5

FIG. 22.


COLUMN TESTS.
Rolled H-Sections.

Vol. XXXV. JANUARY, 1909. No. 1.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

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PAPERS AND DISCUSSIONS

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THE ACTION OF FROST ON CEMENT AND CEMENT MORTAR, TOGETHER WITH OTHER EXPERIMENTS ON THESE MATERIALS.

By Ernest R. Matthews, Esq.,* and James Watson, Esq.

To be Presented March 3d, 1909.

This paper describes in detail a series of experiments, extending over the past two years, made by the writers, in order to ascertain:

The effects of frost, and alternate frost and thaw, on the tensile strength of cement and cement mortar when mixed with—

(*a*) fresh water, cold or warm,

(*b*) sea-water;

The temperature below which it is detrimental to mix Portland cement concrete;

The effects produced by immersing concrete in—

(*a*) fresh water, hard or soft,

(*b*) sea-water;

Absorption of water by (dry) cement;

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

* Borough Engineer of Bridlington, England.

Quantity of water required to be added to cement to produce complete hardening.

The results of these experiments are not only interesting, but should prove of value to engineers generally.

The Hull Cold Storage Company, of Hull, England, kindly allowed the writers to use its refrigerating rooms in order to obtain the degrees of frost necessary for these experiments. The cement used was that manufactured by Robson's Cement Company, of Hull, and all the experiments were carried out at the laboratories of that firm, in Hull, these being kindly placed at the disposal of the writers.

Cement Used in Experiments.

The particulars of the cement used are as follows: Made on January 28th, 1907. Residues on a 5 776-sieve (that is, having 76 meshes per linear inch) = 0.5%; residues on a 10 000-sieve (having 100 meshes per linear inch) = 2.0%; residues on a 32 400-sieve (having 180 meshes per linear inch) = 11.5% (showing that the cement was ground extremely fine). Specific gravity = 3.112; flour = 54.5%; Le Châtelier tests, expansion = 2.7 mm.; Faija bath test, cement pat sound and hard; time of set of neat cement with 25% water: Initial set = 35 min.; permanent set = 6 hours, in a room kept at a temperature of 60° Fahr.

Tensile Strains. Neat Cement.

7 days	= 685 lb. per sq. in.	British Standard Test,	400 lb. per sq. in.
14 "	= 787 " " "	" " "	....
28 "	= 875 " " "	" " "	500 " " "

It will be seen that these results are well above the British Standard Tests.

Tensile Strains. One Part Cement and Three Parts Sand.

Mixed with three parts by measure of sand, and with no hammering of briquettes into moulds, its mean strength was:

7 days	= 200 lb. per sq. in.	British Standard Test,	120 lb. per sq. in.
14 "	= 277 " " "	" " "	....
28 "	= 333 " " "	" " "	225 " " "

Chemical Analysis.

	Percentage
Insoluble residue	0.82
Silica	20.43
Alumina	9.10
Oxide of iron	1.95
Lime	62.65
Magnesia	1.25
Sulphuric anhydride	1.38
Loss on ignition	1.66
Alkalies and loss	0.76
	100.00

The sea-water used in these experiments was taken from the North Sea; the fresh water was drawn from the Hull Corporation mains. To obtain the soft water, the temporary hardness of this water was removed, the permanent hardness being from 3 to 4 degrees.

The experiments herein will be compared with the tests just given, which will be referred to as the "Normal Tests."

Effects of Frost.

The effects of frost, and alternate frost and thaw, on the tensile strength of cement and cement mortar when mixed with (*a*) fresh water—cold or warm, (*b*) sea-water; and the temperature below which it is detrimental to mix Portland cement concrete, were determined by the following experiments:

Experiment A.—In this experiment the writers set out to discover the weakening effect, upon freshly mixed cement, of continuous light frost, temperature 29° Fahr., and of heavy frost, 15° Fahr. Nine briquettes were made with neat cement, 20% water, in the laboratory, the temperature of the air being 60° Fahr. These were taken from the moulds 24 hours after gauging and placed in cold stores, temperature 29° Fahr., and were broken at 7 and 28 days, respectively, the average tensile strength being:

At 7 days = 610 lb. per sq. in.
At 28 days = 905 " " " "

These results are compared with the normal tests as follows:

Tensile Strength, in Pounds per Square Inch.

	Normal tests, 24 hours in air at 60° Fahr., in water remainder of time.	In air at 60° Fahr. for 24 hours then 29° Fahr. for remainder of time.
7 days.....	685	610
28 days.....	875	905

In the 7 days' test it will be observed that there is a decrease of 10.9% in tensile strength, and an increase of 3.4% in the 28 days' test.

Experiment B.—Nine briquettes, made in the same manner as in Experiment *A*, were placed at 60° Fahr. Three were taken out at the end of 2 days, and placed in cold storage for 25 days; three more, at 7 days, for 21 days, and the other three, at 14 days, for 14 days. All were broken at 28 days, the result being as follows:

Tensile Strength, in Pounds per Square Inch.

	Normal test in air, 60° Fahr. for 24 hours, then in water.	In water at 60° Fahr. for 2 days, then in air at 29° Fahr.	In water at 60° Fahr. for 7 days, then in air at 29° Fahr.	In water at 60° Fahr. for 14 days, then in air at 29° Fahr.
28 days.....	875	912	977	942

Experiment C.—Nine briquettes, made as before, were allowed to harden in air for 7 and 28 days at 60° Fahr., the result being:

Average tensile strength at 7 days = 443 lb. per sq. in.
" " " " 14 " = 525 " " "
" " " " 28 " = 775 " " "

Days.	7, 14, and 28 days in air at 60° Fahr.	24 hours in air at 60° Fahr., then in water.
7	443	685
14	525	787
28	775	875

Experiment C_1.—

Sand and Cement Tests (3 to 1): Same as Experiments *A*, *B*, and *C*.

Days.	Normal test.	*A*.	*B*.	*C*.
7	200	163	.....	220
14	277	270	.....	250
28	353	342	*B*2. 317 *B*7. 305 *B*14. 243	322

Experiment D.—Effect of Alternate Frost and Thaw.—The briquettes were allowed to remain for 24 hours under damp flannel, then

in water for three days (60° Fahr.), then in water at the cold stores (temperature varying from 29° to 60° Fahr.). The briquettes were changed every three days.

Days.	Neat.	3 to 1.	Normal.	
			Neat.	3 to 1.
14	787	252	787	277
28	813	323	875	353

Experiment E.—This test was the same as *A* or *D*, but the briquettes were gauged with warm water; temperature, 100° Fahr.

Days.	Neat.	3 to 1.	*A* Test.	
			Neat.	3 to 1.
7	352	133	610	163
14	705	205	....	270
28	728	230	905	342

Experiment F.—(*Salt-Water Immersion*).—Nine briquettes were mixed with fresh water, and, after 24 hours, were immersed in sea-water, and broken at 7, 14, and 28 days.

Days.	Neat (20% water).	3 to 1 (10% water).	Normal.	
			Neat.	3 to 1.
7	770	242	685	200
14	742	278	787	277
28	812	360	875	353

Experiment G.—Nine briquettes were mixed with sea-water (same test as before).

Days.	Neat.	3 to 1.	Normal.		Mixed with Fresh Water and Immersed in Sea-Water.	
			Neat.	3 to 1.	Neat.	3 to 1.
7	693	180	685	200	770	242
14	775	287	787	277	742	278
28	773	293	875	353	812	360

Initial set, 9 min.; final set, 6 hours.

Experiment H.—Nine briquettes were mixed with sea-water, and, after 24 hours under damp flannel, were immersed in fresh water for the remainder of the time.

Days.	Neat.	3 to 1.	Normal.	
			Neat.	3 to 1.
7	628	150	685	200
14	733	255	787	277
28	718	297	875	353

Experiment K.—Same test as *A*, but the briquettes were kept in a temperature of 15° Fahr. in cold storage.

K.—Heavy Frost.			*A.*—Light Frost.		Normal.	
Days.	Neat.	3 to 1.	Neat.	3 to 1.	Neat.	3 to 1.
7	405	57	610	163	685	200
28	595	145	905	342	875	353

The briquettes were taken from the cold stores to the laboratory, two miles away, and were broken in a temperature of 60° Fahr., 45 min. after leaving the cold stores.

Experiment L.—The briquettes, 24 hours after gauging, were put into water at 60° Fahr. for 6 days, then placed in the cold stores at a temperature of 15° Fahr. for the remainder of the time.

Days.	Neat	3 to 1.	Briquettes placed in air at 60° Fahr. for a day or two after having been in cold stores at 15° Fahr. for 28 days.		*A.*	
			Neat.	3 to 1.	Neat.	3 to 1.
28	700	217	875	315	905	342

Experiment M.—The briquettes were put directly into the cold stores at 29° Fahr. for 7 days.

Days.	Neat.	Normal—Neat.
7	480	685
28	595	875

Experiment N.—The briquettes were made with neat cement, and placed, some in air at a temperature of 60° Fahr., and others in air at a temperature of 29.3° Fahr. (2.7° of frost). In 15 min. those in air at 60° Fahr. were still soft, while those subjected to frost had just frozen hard at the expiration of that time. Briquettes mixed with sand and cement (3 to 1) were subjected to a similar test. At the expiration of 15 min. those in a temperature of 60° Fahr. were still soft, while those in a temperature of 29.3° Fahr. had just frozen hard; and at a temperature of 27° Fahr. (5° below freezing point, Fahr.) were frozen very hard indeed at the expiration of that time.

Conclusions from the Foregoing Experiments.—These investigations have led the writers to the following conclusions:

(1) That light frost occurring 24 hours after the cement has been gauged, as indicated in Experiment *A* (3° of frost, or thereabouts), is detrimental to freshly mixed Portland cement, but only for a short time, and that at the end of 28 days it has quite regained its normal strength. If the frost occurs immediately after the cement has been gauged, the effect is more detrimental, and would appear to be permanent (see Experiment *M*). A minimum quantity of water should be added in frosty weather.

(2) That heavy frost (17° of frost, or thereabouts) has a most injurious effect (permanent) upon freshly mixed cement (neat), and cement mortar, as shown in Experiment *K*.

(3) That a light frost (3° of frost, or thereabouts), as indicated in Experiment *A*, does not affect cement or cement mortar if it has attained 2 days' set previous to the occurrence of the frost (Experiments *B*, *C*, and *D*).

(4) That the detrimental effect of light frost upon cement mortar (3 to 1) occurs more immediately than upon neat cement, but that cement mortar recovers from the ill effects of frost more rapidly than neat cement. At the end of 14 days it has quite recovered (Experiment *C*).

(5) That the mixing of cement or cement mortar with warm water (temperature, say, 100° Fahr.), which is sometimes done in frosty weather, and has been recommended by some engineers,* has a permanently injurious effect upon cement and cement mortar. This will be seen by reference to Experiment *E*.

* *Minutes of Proceedings*, Inst. C. E., London, Vol. CXXXIV, p. 384.

(6) That neat cement immersed in fresh water immediately after becoming set, and remaining in water, has a much greater tensile strength than when remaining continuously in air (Experiment *C*), the former being 55% stronger than the latter at the end of 7 days, 50% stronger at 14 days, and 11.3% stronger at 28 days. In 3 to 1 cement mortar, however, the result is different, the tensile strength being practically the same under both conditions.

(7) That it increases temporarily the tensile strength of cement and cement mortar to immerse them in sea-water 24 hours after gauging, instead of immersing them in fresh water; but the increased strength is only temporary (Experiment *F*).

(8) That there is an immediate reduction in the tensile strength of briquettes mixed with sea-water and immersed in sea-water (Experiment *G*) over those mixed with fresh water and immersed in sea-water (Experiment *F*). At the end of 14 days the strength of the former equals that of the latter, but after that period a depreciation again sets in, so that at 28 days there is a deficiency of 5% in the tensile strength.

(9) That the initial set of briquettes mixed with sea-water is 9 min.; permanent set, 6 hours. When mixed with fresh water, the former is 35 min.; the latter, 6 hours (Experiment *G*).

(10) That cement and cement mortar mixed with sea-water, and immersed in fresh water 24 hours afterward, have less tensile strength than when mixed with fresh water and immersed in fresh water (Experiment *H*).

(11) Experiment *L* shows that 17° of frost for 28 days does not kill the process of hardening in the briquette, but only delays it.

(12) It would appear from Experiment *N* that it is detrimental to concrete to mix it when the temperature is below 29.3° Fahr. (2.7° of frost), that being the freezing point of cement and concrete.

The Effects Produced by Immersing Concrete in Fresh Water (Hard or Soft), and in Sea-Water.

Neat Cement.—Neat-cement briquettes (mixed with 20% of fresh water) were immersed in fresh water (both hard and soft) and in sea-water after being in the moulds for 24 hours. In each case the absorption finished at the end of the seventh day: (1) The water absorbed by the briquettes in hard water at the end of the seventh

day = 3.46%; (2) by the briquettes in soft water = 2.92%; (3) by the briquettes in sea-water = 3.92 per cent.

Sand and Cement, 3 to 1.—As in the case of neat cement, just described, no absorption took place after the seventh day. (1) The briquettes in hard water absorbed at the end of the seventh day 3.4% of water; (2) in soft water, 3.4%; (3) in sea-water, 4.5 per cent. In the case of neat cement, and sand and cement briquettes, similar to the foregoing, but exposed to air for 7 days previous to immersion in water, the following results were obtained:

With neat cement during 7 days' exposure to air, the loss in weight was 1.7%; then, after 24 hours' immersion in (1) hard, (2) soft, and (3) sea-water, the increased absorption = 1.7%, just counterbalancing the previous loss. On continued immersion a further gain of only 0.3% in weight was obtained. The results of tensile tests at 28 days were: (1) = 875 lb.; (2) = 760 lb.; and (3) = 835 lb. per sq. in. The lower absorptions were most probably due to the partial set of the cement in the center of the briquettes during the 7 days' drying in the air, and the prevention of the free percolation of the water through it.

With sand and cement, during 7 days in air, the loss of moisture was 2.2%; then, after 24 hours' immersion, the gain was: (1) in hard water = 3.4%; (2) in soft water = 2.0%; and (3) in sea-water = 2.8 per cent. The results of tensile tests at 28 days were: (1) = 360 lb., (2) = 360 lb., and (3) = 330 lb. per sq. in.

Conclusions from the Foregoing Experiments.—(1) That no absorption takes place in neat cement, cement mortar, or concrete after immersion in water for 7 days.

(2) That neat cement immersed in hard fresh water for 7 days absorbs 0.54% more than when immersed in soft fresh water for the same period.

(3) That if immersed in sea-water for 7 days it absorbs 0.46% more than when immersed in hard fresh water, and 1.0% more than when immersed in soft fresh water.

(4) That cement mortar (3 to 1) absorbs the same quantity when immersed in either hard or soft fresh water, but when immersed in sea-water it absorbs 1.1% more.

(5) That neat cement and cement mortar, when immersed in hard fresh water, have practically the same absorption in both cases.

That neat cement immersed in soft fresh water absorbs 0.48% less than cement mortar (3 to 1), and that neat cement immersed in sea-water absorbs 0.58% less than cement mortar.

(6) That when neat cement is exposed to air for 7 days after gauging it loses 1.7% of its weight, but when immersed for 24 hours in water, either hard or soft fresh water, or sea-water, it absorbs 1.7%, which just counterbalances the previous loss.

(7) That after this loss has been made good, and the immersion of the briquettes is continued, there is a further gain of 0.3%, which averages 1.43% less than the absorption which takes place when cement is immersed in water 24 hours after gauging.

(8) That cement mortar (3 to 1) exposed to air for 7 days after gauging loses 2.2% of its weight; but after 24 hours' immersion in hard fresh water the absorption is increased to 3.4%; in soft water it is 2.0%, and in sea-water it is 2.8%, showing that soft water is absorbed more slowly than hard water, and that sea-water is also more slowly absorbed than hard fresh water.

Absorption of Water by (Dry) Cement.

Experiment 1.—Dry cement was compressed into moulds with a press, the same amount in each mould. The moulds were then allowed to stand in water for 24 hours. The result was an average increase in weight of 14.9 per cent.

Experiment 2.—The cement in the moulds was then changed so that there would be a variation in thickness of $\frac{1}{8}$ in. The result was that the absorption varied correspondingly from 13.2 to 17.1 per cent. The writers then took a larger quantity of cement and compressed it into the same bulk as at first, and found that the absorption after 12 hours = 12.3%; after 24 hours = 14.4%; after 48 hours = 14.8%; after 4 days = 15.1%; after 8 days = 15.4%; after 31 days = 16.8% (constant).

Conclusions from the Foregoing Experiments.—(*a*) That when neat cement is immersed in water for 24 hours it absorbs from 14.4 to 14.9%, according to the thickness of the cement;

(*b*) That the absorption increases to 16.8% at the end of 31 days' immersion;

(*c*) That no absorption takes place after 31 days' immersion.

Quantity of Water Required To Be Added to Cement to Produce Complete Hardening.

According to the formula laid down by Le Châtelier,*

$$3CaO\ SiO_2 + \text{water} = CaO\ SiO_2\ 2\tfrac{1}{2}\ H_2O + 2Ca\ (OH)_2$$

35.4% of water would be required to be added to cement for complete hardening, and the hardened cement, therefore, would contain 26.2% of water in a combined state. From the figures the writers now give, this is found to be by no means the case. Possibly the reaction is not so complete, as Le Châtelier works it out by theory, and, although more water is combined by using an excess in gauging, yet the test becomes less and the strength is impaired, for Dr. Michaelis† found that, by sealing up some very finely ground cement with 150% of water for four weeks and then drying it over sulphuric acid, a loss on ignition was shown which equalled 27.5% of water absorbed. The results of the writers' experiments are as follows:

Experiment A.—Neat-cement test pieces gauged with 20% water, after 14 days' immersion in water, gave free moisture = 11.75%, combined = 9.7%; after 28 days' immersion they gave free moisture = 12.14%; combined = 10.9 per cent.

Experiment B.—Mortar briquettes, with 3 sand to 1 cement, gauged with 10% of water, after 24 hours were immersed in water and kept there for 7, 14, and 28 days:

	Free water at 105° cent. Percentage.	Combined water. Percentage.
At 7 days	9.44	9.20
" 14 "	8.49	10.58
" 28 "	8.68	10.29

Experiment C.—Neat cement test pieces were gauged with 20% of water, and, after 24 hours under damp flannel, were immersed in fresh water for periods up to 18 months.

	Free water at 105° cent. Percentage.	Combined water. Percentage.
After 3 days	8.0	6.4
" 7 "	8.9	7.9
" 28 "	9.0	12.38
" 3 months	9.25	13.05
" 6 "	9.55	12.95
" 12 "	9.40	12.90
" 18 "	9.82	12.20

* *Annales des Mines*, 1887.

† *Thonindustrie-Zeitung*, 1899.

Experiment D.—Other test pieces, made of different cement, were also tried, 20% of water being used for gauging, with the following results.

	Free water. Percentage.	Combined water. Percentage.
After 24 hours.....	8.5	3.3
" 48 "	7.45	11.5
" 72 "	7.10	7.7

Experiment E.—Frozen Test Pieces.—The cement was gauged with 20% of water, and, after 24 hours under damp flannel, was placed in cold storage at 15° Fahr. (17° of frost) for 6 days. The free water = 10.29%; combined water = 5.92 per cent. There was no active hardening of the cement.

Conclusions from the Foregoing Experiments.—That from the quantity of water absorbed by cement and cement concrete the writers are unable to determine precisely its ultimate effect upon the hardening of the concrete. It depends entirely upon physical conditions, such as the porosity of the concrete. When an excess of water is added, there seems to be a loss due to evaporation during the few hours after gauging; then, although immersed in water, the percentage of uncombined water remains constant, while the percentage of combined water increases until it attains a maximum of approximately 12 per cent. This occurs also with gauged cement exposed to open air, the only difference being that the quantity of free moisture gradually diminishes, owing to evaporation. In the case of sand and cement mortar, more water is absorbed in proportion to the quantity of cement used; this, most probably, is due to the greater porosity of the mass. The same result, it will be noted, occurs in the case of cement compressed into moulds, less water being absorbed in the same period when the mass is denser in bulk; so that, in the case of fire-proof floors and reinforced work, the denser the concrete, and with no more than the maximum quantity of water used to obtain a perfect gauging, the more fire-proof will be the mass, and less subject to dilatation through the expansion of unnecessary water. In the case of the test pieces subjected to hard frost, the process of hardening has been stayed, and, although the proportion of water used was the same as in Experiment *C*, yet in the same period of time 20% less

of the water had gone into combination, preventing the cement from attaining its normal strength.

The writers have endeavored to present what, in their opinion, is useful information concerning the behavior of Portland cement, cement mortar, and concrete, under various conditions. This information has been obtained by carrying out the series of experiments described herein, in which 727 briquettes were broken.

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THE BONDING OF NEW TO OLD CONCRETE.

By E. P. Goodrich, M. Am. Soc. C. E.

To be Presented March 3d, 1909.

The necessity, in his general practice, of securing as perfect a bond as possible between masses of concrete which are deposited in abutting positions, but with longer or shorter intervals between the times of deposit, together with several demands for expert opinion as to the adequacy of the bond secured under certain given circumstances and with certain materials and methods, led the writer to make investigations into the history of the art, and to perform a series of experiments in the laboratory of the Polytechnic Institute of Brooklyn to determine how closely it is possible to approach a truly monolithic condition at such joints, when care is taken in executing the concreting in their vicinity.

Literature of the Subject.

The principal editorial in *The Engineering Record* of August 29th, 1908, discusses this subject with particular acumen. The remarks end with the statement:

"To sum up the whole matter in a phrase, the real obstacle militating against the bonding of fresh concrete to old, is dirt. This dirt may be an oily substance or a finely comminuted material forming an

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited, from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and when finally closed, the papers, with discussion in full, will be published in *Transactions*.

inert stratum constituting a most objectionable divided joint, or any similar substance which prevents the new crystals of setting from attaching themselves continuously to those already existing. It is commonly supposed and stated as a general proposition that new concrete cannot be bonded to old. There may be room for legitimate difference of opinion as to the practicability of attaining that end. Any process not feasible of execution under ordinary conditions of work may be so difficult as to be practically impossible, but some engineers of experience believe that bonding new concrete to old will not long be classed in that category. If such a bond be attempted with a truly clean surface experience shows that intelligent manipulation involving some pressure will insure invariably successful results. It has many times been done on a small scale, and it is not too much to expect that under suitable conditions further experience intelligently directed may accomplish the same ends under ordinary circumstances. Efficiency of concrete work will be greatly enhanced in many cases if such an important end can be uniformly attained; and when it can be done on a small scale, ultimate success on the large scale of usual engineering works is not too much to expect."

The use of oil in some form, coated upon the forms so as to prevent the adhesion of any concrete to them, is a direct cause of much trouble when a good bond is desired between the old concrete and any further concrete, stucco, grout, etc. The primary reasons for the use of this oil are:

(*a*) To prolong the life of the forms (the alternate wetting and drying tending to rapid deterioration), although the natural life is not usually long enough to make this item of much importance.

(*b*) To lessen the labor incident to the removal of the forms.

(*c*) To lessen the breakage of the forms due to sticking while being removed.

The handling of oil about any parts of the work is likely to provide sufficient material to add to the natural difficulty of securing good bond at far distant points. Men carry it on their clothes, their shoes, and their hands, and, as it usually floats on water, a thin film is likely to rise to the top surfaces of fresh concrete everywhere and remain on top of the concrete when the excess water evaporates. There is seldom enough alkali in the cement to saponify completely such oil within any appreciable lapse of time, and, even when such saponification does take place, the soaps thus formed are sometimes not readily soluble in water, and there is practically never sufficient water used to dissolve and remove them, even when they are soluble.

For these reasons, the writer has lately made it a practice not to coat his forms, or else to use soap for the purpose; either hard or soft soap is suitable, and the only drawback to its use is its relatively higher cost. Its prime advantage is its solubility in clear cold water. When soap is used, it is necessary merely to flush the surface thoroughly with water to remove the otherwise troublesome film. In securing a bond, the excess water is useful for other reasons, as well.

Laitance is often responsible for imperfect bond. This point is discussed in the following remarks by L. J. Hotchkiss* on "Some Details of Concrete Construction":

"Occasions will arise, however, where a longitudinal joint in a wall is necessary, and where this is the case the thorough cleaning of the concrete is often neglected. The laitance is not all removed and dirt, shavings and sawdust are permitted to remain in the forms. This work cannot be left entirely to laborers but must have efficient supervision and inspection. The surface of the concrete should be scrubbed with wire brushes and water until every stone shows clearly, and the water used in the scrubbing should be flushed off and not left standing in the forms with the laitance in suspension, to be deposited on the concrete again as soon as the scrubbing gang has gone. Both time and expense can be saved if the cleaning is done before the concrete becomes hard. It can be done with shovels and a small amount of washing instead of the protracted scrubbing necessary later. The bond obtained in even the most carefully cleaned joint is of uncertain strength, and if the cleaning is poorly done, there is no bond at all."

Shrinkage cracks almost invariably reveal themselves in reinforced concrete floors along the joints between successive days' work. After a time, retaining and other walls of mass concrete very often reveal even the horizontal joints between successive deposits, and the writer knows of one such wall through which water has found its way to such an extent along such joints as to ruin the appearance of the face of the wall and actually cause anxiety as to its continued stability.

Taylor and Thompson† cite an instance "In the New York Subway", where:

"Work was commenced with no provision for bonding horizontal layers, but it was soon found that more or less seepage occurred, and in one case where a large arch was torn down the division line between two days' work was distinctly seen."

* *The Engineering Record*, March 9th, 1907.
† "Concrete, Plain and Reinforced," p. 376.

In demolishing old concrete work with wedges and sledges, an experienced workman always looks for the joints between successive layers, and the writer has often seen large masses separated by using only a crowbar when the old seam had been discovered. The commonly observed hollow sounds which issue from cement sidewalks and floors under traffic when the floors and walks have been constructed with special cement-finish coats are always due to a separation having taken place between them and the base course. This is obviously due to poor bond. After any considerable lapse of time, stucco, mortar, plaster, and grout which have been applied to mass concrete work, are oftener found to have separated than to have retained a perfect bond with the backing.

Gillette and Hill* call attention to one primary reason why proper bond is difficult to secure on horizontal and other much-worked concrete surfaces, where the mortar has been flushed to the surface:

"Concrete which has set hard has a surface skin or glaze to which fresh concrete will not adhere strongly unless special effort is made to perfect the bond. * * * The secret of securing a good bond between fresh concrete and concrete that has set lies largely in getting rid of the glaze skin and the slime and dust which forms on it."

In a paper on "Construction Details of Reinforced Concrete Work",† De Forest H. Dixon, Assoc. M. Am. Soc. C. E., says:

"The sloping off of concrete in the beams and girders should never be allowed. It is a well-known fact that there is almost no bond between old and new concrete."

and again:

"Granolithic finish should be placed monolithic with the slab, or, if placed afterwards, which is desirable in stables, and some other classes of work, its thickness should never be less than 1½ in. Recently, an acid solution has been placed on the market which the author has tried in several instances, using, in some cases, finish as thin as ½ in., put on concrete that had attained an age of several months. This finish has been in place for three months, and thus far shows a perfect bond."

The specifications prepared for large engineering works to be executed in concrete usually contain a short clause with regard to bonding old and new concrete, but they are very rarely at all explicit.

* "Concrete Construction, Methods and Costs."
† *Proceedings*, Municipal Engineers of the City of New York, 1907, p. 1.

the exact treatment usually being left to the resident engineer for determination. The following is the standard specification for work of this class, incorporated in all the published requirements for the several parts of the new Catskill Water Supply Development for New York City.

"For the proper bonding of new masonry to old, such provision shall be made of steps, dovetails, or other devices as may be prescribed. Whenever new masonry is joined to old, the surface of the old masonry shall be thoroughly cleaned, using stiff brushes and a stream of water; the laitance shall be scraped off and the surface shall be clean and wet at the moment the fresh concrete or mortar is placed. When ordered, a thick wash of rich mortar shall be brushed over the contact surface of the old concrete."

This specification is typical in the main, being rather better than most of the others in the writer's possession.

In the construction of tunnels, and heavy or extensive work of all similar kinds, joints are inevitable, and the designer must study each case carefully and devise proper quantities and arrangements for the reinforcement which is essential to securing the work against the appearance of shrinkage and temperature cracks along such natural joints. The subjects of setting, shrinkage, and change of size, from fluctuations of temperature and humidity, are intimately connected with the subject of bonding old and new concrete, but will not be discussed here, although they have all been investigated, both experimentally and analytically, by the writer in this and closely related work.

Another and a very vital and closely related question, and one which exists in the minds of most constructors, is as to the proper place in reinforced concrete floor-beam constructions at which to leave the joint between successive days' work. In the last analysis, if a perfect bond could be secured between the successively deposited masses, it would not matter in the least how or where one day's work were stopped. Usually, a compromise has been deemed best, and the joint made at about the quarter point in the span of a beam or slab, where neither the bending moment nor the shear is a maximum.

Patented Processes.

All the treatises on concrete discuss this important subject of bonding, but tell very little of real value. Ernest L. Ransome, Assoc.

Am. Soc. C. E., has secured several patents on the general treatment of concrete surfaces, with mechanical bonding devices and with acids, to secure a better connection; and other methods have been devised which make use of bituminous and other compounds to be applied to the old work and in preparing the new mortar. These are mostly used where it is the intention to apply stucco or plaster to mass concrete. Usually, these compounds are also claimed as resisters of dampness as well as bonding agents. Only one such process, not primarily for damp-resisting purpose, is known to the writer. In it a special preparation of coal-tar is used in the place of water in mixing the mortar for the new floor or sidewalk wearing surface.

The first patent (No. 647 904), issued to Mr. Ransome, concerning the subject of bonding, was dated 1898, and covers a "concrete joint strengthened with a metallic coil embedded in both of the joined masses."

Patent No. 694 578, also issued to Mr. Ransome, covers:

"The method of forming a bond between two bodies of concrete, which consists in first sprinkling over the surface of the first made body of concrete while yet soft and unset, a layer of prepared honeycomb slag, embedding the same into the surface, allowing the concrete to set and after removing all superfluous slag, covering it and embedding its exposed surfaces with the second body of concrete."

Patent No. 767 582, issued to Mr. H. L. Lewman, is

"In a concrete construction, adjacent sections connected by a bendable tie-piece having looped portions embedded in each section with its free ends meeting at its mid-length, and a connecting strip extending between said looped portions of the tie-pieces and having its free ends bent to intersect the body thereof, or with a rod extending through the opposite loops of each tie-piece at an angle thereto."

The best known of these patents is No. 800 942, issued to Mr. E. L. Ransome in 1905. It has to do with:

"The improvement in the art of forming structures of concrete and the like, which consists of forming a layer of plastic material containing cement, and permitting the same to harden, in removing portions of the hardened surface of said layer by the application of an acid thereto, in removing from said surface by washing the excess of acid and resultant salts and disintegrated material, and in forming a second layer of plastic material containing cement, over said surface of the first layer, and permitting the second layer to harden, whereby to form a perfect bond between the two layers."

In a pamphlet describing the use of a special mortar for cement floors, stucco, etc., Albert Moyer, Assoc. Am. Soc. C. E., gives the following method of securing proper adhesion between old and new cement in cement floor work:

"Sweep off the concrete surface thoroughly with a broom, wash the surface with clear water and sweep off with a broom. As soon as the excess water has been taken up by the atmosphere or absorbed in the concrete, paint the surface with a solution of one part commercial muriatic acid, three parts water; paint on three coats one right after the other. Within a few minutes wash off the surface with clear water, sweeping with a stiff broom. This removes all particles of dead cement which may remain on the surface, exposing each grain of sand and other aggregates which compose the concrete in very much the same condition as they were before being mixed. This also wets the concrete quite thoroughly.

"While the concrete is still wet, place an inch coat of mortar * * * ."

The removal of efflorescence from concrete work, by the application of an acid wash, is so similar in method and result to that required in obtaining a perfect bond that the following description* may well be included for the sake of completeness:

Commercial hydrochloric [muriatic] acid, diluted with from 4 to 5 parts of water, was applied, and the surface scrubbed with common floor scrub brushes, water from a small hose being constantly played on the concrete while the cleaning progressed, so as to prevent the penetration of the acid. The cost on straight work was estimated at about 2 cents per sq. ft. cleaned. An experienced stone renovator should be employed whenever possible. Hydrochloric, acetic, and oxalic acids were experimented with, in conjunction with the scrubbing. All were effective. Hydrochloric acid was the best, requiring less scrubbing. Acetic acid came next in efficiency.

In this case the endeavor was simply to remove the thinnest possible layer from the surface, the acid being prevented from penetrating to any appreciable extent. In the patented process for securing decorative surface results on concrete by etching with acid, covered by letters patent No. 716 371 granted to Block and Richards, December 23d, 1902, the method is closely analogous to that necessary to secure a good bond, and is described as follows:

"The process of producing a natural finish on artificial stone composed of cement and particles of natural stone by subjecting it when

* Abstracted from *Engineering News*, October 31st, 1901.

molded and dried to a solution of muriatic acid, then rinsing the same in water, then subjecting the same to a solution of carbonate of soda, and finally rinsing the same in water."

Published Experiments.

Few experiments have thus far been published with regard to methods of bonding masses of concrete, but in *Engineering News* of August 13th, 1908, is given an abstract of a graduation thesis prepared by Raymond B. Perry, of Case School of Applied Science. He prepared and tested by flexure small concrete-beam specimens, $2\frac{3}{4}$ in. square and $27\frac{3}{4}$ in. long, which had been spliced at their centers. A variety of methods of splicing were used, none of which developed the strength of an unspliced specimen. His results and deductions are repeated here for future reference and comparison.

TABLE 1.—Perry's Tests.

Condition of Joint Surface.	Roughened.	Smooth.	Rough.	Smooth.	Moulded Groove.	No Joint.
Treatment.	Wet.	Grouted.	Grouted.	Ransomite.		
Results of tests, in pounds per square inch	158	119	279	232	173	255
	123	131	211	128	133	230
	87	207	237	220	120	249
	124	82	225	231	128	271
	128	124	227	146	110	257
Averages, in pounds per square inch...	124	133	236	191	123	252

"While experiments of so limited character are not absolute in their results, it may be concluded with some degree of certainty:

"(1) That the bond existing between new mortar or concrete and old, where the old surface is smooth, is very slight.

"(2) That about one-half of the strength of the concrete is developed in a joint bonded (*a*) by roughening the old surface; (*b*) by applying a layer of cement paste; (*c*) by providing the old surface with a bonding groove.

"(3) That a large part of the strength of the concrete, perhaps as much as 90%, is developed where the old surface is roughened and a layer of cement paste is applied.

"(4) That such a solution as 'Ransomite' practically takes the place of the roughening, since a bond made with it is otherwise similar to the one made in these tests by roughening the old surface and applying a layer of cement paste."

"Ransomite" is a powder which is to be dissolved in water in order to obtain the bonding solution.

These tests closely followed some experiments described in *Annales des Ponts et Chaussées*, Pt. 3, Vol. 27, 1907, the results of which were summarized as follows in *Engineering News* of December 12th, 1907:

"A [neat] cement joint is an aid in connecting old concrete work to new, and * * * whether this joint is used or not, an excessive tamping of the new work near the joint will add appreciably to the strength of the connection."

TABLE 2.—FRENCH TESTS.

SPLICED AFTER:	7 DAYS.		30 DAYS.	
Treatment of joint.	No grout.	Grouted.	No grout.	Grouted.
Results of tests, in pounds per square inch	24 97 67	269 175 300	171 154 142	159 302 227
Average, in pounds per square inch.....	63	223	156	229

It is stated that the above splices, made after 30 days, were better tamped than the earlier ones. It is also to be noted that the mechanical arrangments of the test brought a considerable direct compressive stress upon each joint, and in consequence these tests are not strictly comparable with any others.

The specimens were of the same size and kind as those described above, and all are open to the objection of revealing results comparable only with one another, as there is at best only an approximate relation between the maximum stresses developed in bending and those produced under direct stress.

Professor Morsch* gives the following results, showing that the direct stress may be considered to be about one-half that developed in bending when computed according to the usual formula. The stresses are in kilograms per square centimeter.

TABLE 3.—MORSCH'S TESTS.

Mixture.	1 : 3		1 : 4		1 : 7	
Percentage of water.....	8	12	8	14	8	14
Bending stress.....	21.4	23.2	16.1	16.7	18.3	12.8
Direct tension.....	12.6	10.5	9.2	8.8	4.4	5.5

* "Eisenbetonbau," pp. 35-36.

The writer is not aware of any experiments having been made with regard to the bonding qualities of stucco or cement finish for concrete floors or sidewalks, except a single test with brick as a backing, made by Professor Ira H. Woolson, at Columbia University, for the exploiters of a special water-proofing cement mortar. His report reads in part as follows:

"Description of Specimens.—Two specimens were submitted for test, and each consisted of a common red building brick, with a coating on one side of about one-half inch of the cement. The cement was thoroughly dry and hard.

"Object of the Test.—The object of the test was to ascertain what degree of heat the cement would sustain without injury, either by cracking or by disintegration.

"Method of Test.—In order to develop the full effect of possible differences in coefficients of expansion due to heat in the two materials, the specimens were heated on the cement side only, and the brick kept cool as it would naturally be in a wall, where the face of the wall was heated. To accomplish this object, the specimens were laid, one at a time, horizontally before a side opening in a small gas blast furnace. To insure that the heat be uniformly distributed over the whole face of the cement, a $\frac{3}{16}$-inch sheet of composition asbestos board was used to close the furnace opening, and the cement face was placed near to it, but not touching.

"In this narrow air space between the cement and board the thermo-couple of a Le Châtelier electrical pyrometer was placed, by which the temperature of the face of the cement would be accurately and continuously recorded.

"The heat was raised gradually and temperature read every three minutes. The total time of heating the first specimen was forty-five minutes, and the temperature attained was 1 000 degrees, F. At the time no cracks or other defects were noticeable in the cement, and the brick was cool enough to allow its being easily removed from the furnace by the bare hand. However, in a few minutes after removal, a crack developed along the line of juncture, between the cement and brick, which gradually spread until it had separated about half of the cement from the brick.

"The cement adhered to the brick so firmly that the crack in places ran into the body of the brick itself. No other disintegration occurred.

"The second specimen was heated the same way, but the time of heating was reduced to fourteen minutes, and the temperature to 600 degrees, F.

"This specimen showed no effect whatever due to the heat either hot, or later when it had fully cooled.

"It is quite evident that the cracking of the first specimen was due to unequal expansion of the brick and cement, and rapid cooling. It is possible that the cracking might not have occurred if the specimen had cooled slowly."

While the engineering periodicals have published quite a number of communications from time to time with regard to the proper points in concrete floor construction at which to stop work, still these discussions have all been of an entirely academic nature, no experiments having been made with regard to this fact, so far as known to the writer, except in the case of the reinforced concrete reservoirs for Mexico City, described by James D. Schuyler, M. Am. Soc. C. E.*

There are four reservoirs, all circular in form, and 105 m. in diameter at the top. The roof consists of a 6-in. slab of reinforced concrete supported by radial and concentric beams, resting on cylindrical pillars of concrete, all properly reinforced.

"In the building of such a massive roof over so large an area as each of these reservoirs presents, the question of sustaining the forms during the time required for the setting of the concrete was one of the most serious problems to be solved. To build up timber supports of requisite stiffness and stability reaching from the floor to the roof, on account of the height, would have required a vast forest of timber, of little subsequent utility, and costly to erect and remove. It was finally determined to suspend the forms from the tops of the columns by a series of steel I-beams of standard sizes and lengths which would have a market value for building construction of at least 75 per cent. of their cost after the works were completed. But in order to place the top series of I-beams high enough to act as suspensors for the forms of the main reinforced concrete beams, it was necessary to build up the capitals of the columns to the ultimate height of the top of the roof as bases on which to rest the temporary hanger beams, and in these concrete capitals, which form the ends of four intersecting concrete beams, mold all the steel reinforcing rods subsequently required for the concrete beams. This in effect meant the formation of the ends of the beams a long time prior to their ultimate completion. Hence to determine the effect of this composite construction of a beam, with a joint near each end, upon the final strength of the beam, a series of tests were made in the Condesa testing laboratory, with four full-size beams reinforced as they are to be used in the roof of the reservoir. Two sets of tests were made, with two beams each, 14 and 28 days after molding. These beams were 14 ft. 9 in. long, 19⅝ in. deep, 7⅞ in. wide, with a clear span of 13 ft. 1 in. between supports. In

* *The Engineering Record*, March 28th, 1908.

one beam of each set, the two ends were molded two days in advance of the remainder, in the form shown by the shaded areas of the diagram, leaving a well-defined line of jointure between the concrete of the ends and the central portion of the beam.

"Test A, at 14 days, was made under the following conditions: The solid beam had been molded complete at one operation, with 1:2:3 concrete, reinforced at bottom with two square corrugated bars ½ in. by 14 ft. 10 in.; two diagonals ½ in. by 15 ft. 6 in.; three oblique bars ½ in. by 5 ft. 2 in. at each end, and diagonals, from center to ends, ½ in. by 7 ft. 6 in., the metal in all being 0.7 per cent. of the beam. With 3 809 kilos applied at two points at one-half meter from central axis, the deflection was 2.4 mm.; with 6 185 kilos applied load, the deflection was 4.6 mm.; with 10 369 kilos the deflection was 8.2 mm., when three cracks appeared, half the depth of the beam, at distances of 1.5, 2.9 and 3 meters from one end. The final rupture occurred under a weight of 11 460 kilos.

"The second beam of the pair, molded in sections as described, showed a deflection of 2.4 mm. under 3 908 kilos, 4.8 mm. under 6 815 kilos and 8.0 mm. under 10 369 kilos, when four cracks appeared, 20 cm. long, at distances of 1.5, 1.7, 3.0 and 3.1 meters from one end, all inclining toward the load. The final rupture took place with a deflection of 20 mm. under a load of 13 648 kilos without the slightest sign of weakness appearing at the joints in the concrete. * * *

"Test B, at 28 days, gave results quite in parallel with those of Test A. The mixture was the same, but the reinforcing metal was greater, amounting to 1.0 per cent. of the beam, consisting of three straight base bars in bottom, two of which were ½ in. square and one ⅝ in. square, 14 ft. 10 in. long; also two base bars with diagonal to top at ends, each ½ in. square and 15 ft. 6 in. long; three intermediate top and diagonal bars at each end, each ½ in. square and 5 ft. 2 in. long and two central diagonal and top bars, each ½ in. square and 7 ft. 6 in. long.

"The beams showed the following comparative results during test:

"Applied Load. Kilos.	Solid Beam. Deflection.	Beam Molded in Sections. Deflection.
3,809	2.0 mm.	1.8 mm.
6,815	3.0 mm.	3.2 mm.
10,369	7.0 mm.	5.6 mm.
12,556	10.0 mm.	7.2 mm.
14,742	13.0 mm.	8.8 mm.
16,929	22.0 mm.	10.2 mm.
10,018	Ruptured by compression.	
19,116	.....	12.4 mm.
21,299	.....	16.4 mm.
22,390	.....	Ruptured by compression.

"In each beam, cracks began to appear under a load of 10 369 kilos. * * *

"In this case, also, the absence of any cracks or sign of weakness on the line of joints between the older and newer portion of the concrete beam seemed to prove conclusively that there was no weakness to be feared from this fragmentary method of molding as compared with the monolithic beams, but, on the contrary, they developed greater strength in each case."

The Writer's Experiments.

It did not seem necessary to experiment as to the necessity of a thorough cleaning of the old concrete surfaces. This requirement is so obvious that it is surprising to note the way many other constructors apparently ignore it. A number of years ago, the writer discovered the advantages of using a jet of steam for this purpose, and in *The Engineering Record* of March 7th, 1907, he advocated its use, and gave several other hints on concrete work. The only other engineer who has advocated it is Mr. Ernest McCullough, who writes as follows:*

"Nothing is so absolutely bad for joints as sawdust and nothing is so hard to get rid of. Shavings and blocks of wood are picked up with rag pickers' sticks, which are pieces of wood about one inch square having a sharpened nail driven into one end. Loose gravel, etc., can often be brushed out. Sawdust however remains. Even a strong stream of water fails to get rid of it. Live steam at a high pressure will however clean off the surface of the concrete to the bone. It removes all the half-set and dead cement and all the sawdust.

The writer's experiments were of three varieties, and were arranged with the following objects in view:

(*a*) Measurement of actual tensile strength with five methods of bonding, applied to two varieties of surfaces, and after two different time intervals;

(*b*) Determination of the qualitative effect of exaggerated temperature changes on samples representing cement floor or sidewalk finish, placed after a relatively long lapse of time, with the help of acid treatment, as compared with simultaneous placing of the bottom and top layers;

(*c*) Determination of the advisability of depositing concrete for a deep beam in separate layers with intervening time intervals;

(*d*) Determination of the best point at which to splice concrete floor members between successive days' work;

* "Reinforced Concrete," p. 108.

(*e*) Value of acid treatment in ordinary floor and walk work.

The tension tests were made on standard cement briquettes. Only 1:2 (by weight) Portland cement mortar was tested.

All specimens were cured in air only, to be more nearly analogous to practical working conditions. Splices were made after intervals of 24 hours and of 7 days. All specimens were broken at the end of 30 days. For the sake of comparison, unspliced specimens were moulded and broken after the same period. One series of briquettes was made and broken after 24 hours. The half briquettes thus obtained were utilized in one series of experiments, half of them being spliced after a lapse of 24 hours and the other half after 7 days. It was considered that this series would represent conditions when the surface of old work was entirely broken away and all loose material removed just before the new material was deposited. Another series of briquettes was moulded with a thin strip inserted at the smallest section. When removed from the moulds, of course, these briquettes fell apart into two pieces, the surface next the partition having the exact appearance of having been moulded against a common wooden form. After 24 hours, and 7 days, these half briquettes were spliced, the surfaces to be bonded first being indented with a sharp steel tool in just such a form as would be given with a pickaxe or blunt chisel, according to the usual methods of roughening surfaces to be joined. Five methods were used in making bonds, or rather in connection with the making of the bond:

(*a*) The soft mortar was moulded against the half briquette with only as much manipulation as is usual in depositing concrete work. The old half was used dry in its natural state.

(*b*) The old half briquettes were first soaked in water for about 10 min. and then the new end was moulded as in (*a*).

(*c*) A creamy mixture of neat cement was prepared, and the ends of the half briquettes were dipped in it. This formed a coating similar to that most often specified and used. The new ends were then moulded in the usual way, before the grout had set to any great extent.

(*d*) The ends of the old briquettes were liberally washed with a 1:10 solution of commercial hydrochloric acid. They were then washed in clear water to remove all unspent acid, and the new ends were moulded as usual.

(*e*) A solution of "Bonsit" (a powder similar in nature to "Ransomite") was prepared in accordance with the directions sent with the can, or at the rate of 5 lb. of the powder to 10 gal. of water. This solution was then used in the same manner as in (*d*).

Three specimens representing each age, type of surface, and method of bonding were prepared.

Mortar specimens were decided upon, because it was believed that it is the mortar which has been flushed to the surface and against the forms on each side of the joint, which is the real bonding medium.

Briquettes were used for several reasons:

(*a*) Measurements of the true tensile strength of the bond could be most easily obtained.

(*b*) With the small size of the specimen it was thought that one would be more likely to obtain more uniform results than with specimens of larger size.

(*c*) If good results could be obtained with small specimens, a standard would be provided to be applied over larger areas.

There may be some question as to the truth of Assumption (*b*), because, over larger areas, local deficiencies would be merged with strong sections, so that a somewhat smaller variation would probably be obtained. The actual wide variation, in some cases, is instructive as to the probable conditions at different points in any joint. The narrower range found with some methods of bonding demonstrates, if anything at all, that it is possible with them to gain a better distribution of stress over larger areas than with the other methods.

The powder when put into solution gave a white precipitate when tested with barium chloride, showing that the acid in this case is sulphuric.

The results obtained from the tension bonding tests are contained in Table 4.

While the results in Table 4 are not as concordant as might be wished, still certain deductions may safely be made:

(*a*) The best results were obtained with surfaces having the roughness produced by the natural fracture. This points toward a purely mechanical bond, or one with little cementitious action.

(*b*) Practically no bond was obtained on surfaces which were moulded against forms and had dried for 7 days. This is shown by the large percentage prematurely broken, even with careful handling,

and the relatively low results obtained at best. Thorough soaking of the old work is essential.

TABLE 4.—GOODRICH BONDING TESTS.

	Rough Surface.				Smooth Surface.			
	24 Hours.		7 Days.		24 Hours.		7 Days.	
	Each.	Average.	Each.	Average.	Each.	Average.	Each.	Average.
Plain	161, *C*, 92	126	285, 313, 343	314	162, 205, 80	149	*C*, *H*, 15	15
Soaked	*H*, *H*, 60	60	138, 218, 212	189	165, 90, 208	154	*H*, *C*, 44	44
Grouted	380, 315, *C*	347	347, 280, 200	276	98, 43, 160	100	*H*, *H*, *H*	.....
HCl	68, *C*, 155	112	126, 175, 221	174	87, 232, 105	141	83, 51, 44	53
"Bonsit"	30, 60, *H*	45	152, 105, 95	117	79, 54, 68	67	172, 26, 114	104
Average	147	.....	207	.....	122	.....	69	.....

Whole Briquettes.
363
384
309
438
415
304
427
Av. 377

H = Broken while handling before expiration of 30 days.
C = Broke in testing machine while adjusting clips.

(*c*) It was noted with regard to the 7-day, smooth surface, grouted bond, that the water was absorbed so rapidly from the grout as to render it an actual detriment. However, the results obtained with regard to the 24-hour and 7-day, rough surface, soaked, specimens might lead one to infer that there is a possibility of using too much water. This latter may be the case in laboratory work, but, in the writer's building experience, few laborers have ever been met who were prone to use water to what seemed an excess. In applying stucco work, or lining reservoirs, however, an expert workman knows that there is a real possibility of too completely saturating the old work,

and it requires long experience and great care to obtain exactly the proper humidity.

The use of dry cement sprinkled over old concrete surfaces which have first been dampened is of questionable value, in view of the writer's experience.

(*d*) A coating of grout is advantageous, first having soaked the old concrete thoroughly to prevent the water in the grout from being absorbed too rapidly.

(*e*) Acid treatment at an age of 24 hours is not of appreciable value.

(*f*) On surfaces originally moulded against forms, after having dried for 7 days, acid is advantageous.

(*g*) Results with the use of acid were more consistent among themselves than with any other method.

(*h*) Why the splices made after the 7-day interval are so much better than those made after 24 hours on the rough surfaces, is difficult to explain, unless it be that the breaking to obtain this rough surface had loosened many small particles which still clung to the damp surface while only 24 hours old, but which were naturally removed during the handling incident to the 7-day tests.

(*i*) The bonds secured on rough and smooth surfaces after 24-hour intervals are not materially different.

While the acid treatment does not give a bond as strong as when the old surface is rough-pointed, still its cost is much less, and where an absolutely monolithic condition is not essential, but where fairly good bond is required, the use of acid is of value.

Again, where proper reinforcement is provided across joints, the use of acid is advisable and sufficient.

The powdered material is much more convenient to handle than the commercial acids shipped in glass carboys.

In view of the writer's and the other bonding experiments quoted, the following may be given as a proper mode of procedure:

"For connections made after a lapse of 24 hours or more, break back the surface concrete to firm material, and clean the fresh surface with steam, air blast or forceful water streams so as to remove all fine loose material. Saturate well, but not so that water stands on the surface or oozes from the material. Paint completely with neat cement grout, mixed to the consistency of thin cream, just before new concrete is deposited, and see that the latter is of proper mixture, containing a proper proportion of mortar, which should be worked against the joint so as to be certain that no voids exist in its vicinity.

"For connections made after long intervals, so that the old cement has set hard, and where the expense of rough-pointing the whole surface is greater than is required because of the nature of the desired bond, use commercial muriatic acid, diluted with clear water, 1 to 5, or the commercial bonding powders, dissolved in clear water at the rate of 5 lb. of powder to 10 gal. of water. First wet the old concrete surface with so much water that a fresh wetting is not immediately absorbed. Remove any excess of moisture and, when the surface appears as if commencing to dry, paint on the old surface three successive coats of acid one after the other. Let this remain for about 30 min., after which carefully clean the surface of unspent acid, soluble salts, and fine material, with plenty of water, finally cleaning with a steam jet or air blast if obtainable. Just before the fresh concrete is to be deposited, and while the old material is still very damp, apply a thin coat of neat cement grout mixed to the consistency of thin cream, just before the new concrete is deposited, and see that the latter is of proper mixture containing a proper proportion of mortar, which should be worked against the joint so as to be certain no voids exist in its vicinity."

A block representing a small section of a concrete sidewalk, or floor, was also prepared. It was just of a size to fit a furnace door under a high-pressure steam boiler. The concrete base was moulded 3 in. thick, in two operations, each covering half the area of the block. Reinforcing rods were left protruding from the first part deposited, so as to cement it firmly to the second one. The latter was placed 7 days after the first one. The surface of the latter was prepared to receive the finish coat by applying a solution of "Bonsit," according to the specification previously given. As soon as the concrete of the second section had partially set a ½-in. coat of 1:3 cement mortar was spread over the whole area of the block and well troweled so as to secure as good a bond with the backing as possible. The whole was allowed to set for 30 days, and was then subjected to an alternate heating and cooling by being placed first in the furnace door for a few minutes and then allowed to cool in the outdoor air. These alternate periods of heating and cooling were as follows:

Heated 12 min.	Heated 7 min.
Cooled 13 "	Cooled 15 "
Heated 15 "	Heated 5 "
Cooled 35 "	Cooled 15 "
Heated 5 "	Heated 10 "
Cooled 13 "	Cooled.

The temperature on the mortar face (the one toward the fire) was allowed to rise until steam was seen to issue from the edge of the block between the mortar and the concrete. The temperature at the depth of the joint was then approximately 200° Fahr. The exposed face was much higher, while what would be the bottom of the floor or sidewalk was not more than 60° Fahr. when tested by the hand. The block was then removed from the fire and allowed to cool until the hand could rest comfortably upon what had been the exposed face. This would correspond to a temperature of about 120° Fahr. The heating operation was again repeated, the time of exposure again reaching a point when steam appeared. After the second heating, the back of the block reached a temperature which was just a little too hot for the hand. At each cooling this fell to about 120° Fahr., but at the same time the mortar face must have had a range of temperature of at least 100 degrees.

After the second heating, a minute examination of the edges disclosed a hair crack along the joint between the two layers of the oldest and the newest materials; when the block had cooled so that it was about ready for another heating, a fine crack was also observed across the top along the line between the two parts of the backing. Evidently, a separation would have taken place except for the reinforcing bars. This crack could not be found on the bottom of the block, and it closed up entirely during each heating, appearing again during each successive cooling.

After the third heating, a narrow strip around two edges sounded hollow when tapped with a hammer. After each successive heating and cooling, this hollow area increased somewhat, until, after the fifth heating, it included about half the area over the older section. After the final heating, however, when a hammer was used to break away the loosened surface coat, not more than two-thirds could be thus loosened, some parts carrying off adjacent portions of the backing. That portion in which the top and bottom were placed at the same time could not be separated with a clear plane of cleavage, even after the specimen was broken into fragments. In the latter condition the two parts of the other half of the block readily parted under a smart blow. However, from the writer's experience, and from a study of the conditions of the test at the site during its progress, it can be positively affirmed that the value of the acid treatment was demonstrated by the adhesion still remaining even after the repeated heat applications.

No physical or weather conditions are ever as severe as those provided in connection with the furnace, and, even if each heating and cooling might be likened to an annual experience, ample bond remained after five summers and winters to warrant a contractor's guaranty, if the whole bonding operation were faithfully executed. Under actual weather conditions, the changes of temperature are not nearly as sudden or widely varying, so that the effects of the slightly different coefficients of expansion are very small.

On the basis of coefficients of 0.00000655 and 0.00000795 for mortar and concrete, as given by Bonniceau in *Annales des Ponts et Chaussées,* and with a temperature difference of 100° Fahr., there would be a difference in length of only 0.00000140 for each degree, or 0.000140, so that in blocks 100 in. square, in which the maximum probable action would be more than 50 in., the difference would be only 0.007 in. On the assumption of a modulus of elasticity of 2 500 000, the resulting tensile or compressive stress (ignoring any reduction in amount due to resulting strains) would be only 3½ lb. per sq. in. if uniformly distributed, or at that rate if concentrated at any one or more points. Whenever the bond becomes broken it must therefore be concluded that it was not sufficiently strong to withstand a shearing stress between the top and bottom layers of the amount thus computed. When it is recalled that such shearing stresses should be much more than equal to corresponding tensile ones, the inadequacy of much work is apparent, and the tests of the tensile strengths developed by the various methods of securing a proper bond tend to prove that a sufficient bond can be secured in floor and sidewalk work with proper workmanship, particularly if some special method like the acid treatment is used, in the very execution of which a sufficient number of special precautions are added to improve the result perceptibly.

The final experiment consisted in the fabrication of a reinforced concrete beam, doing the concreting in sections, each succeeding part being deposited after a lapse of about a week from the last one, while the beam was tested after a final lapse of only two weeks after the last section was concreted. The several parts and the order of deposit were as shown in Fig. 1. The mixture used was 1:2:4 (by weight), the broken stone being the run of the crusher, up to ¾-in. One of the bonding powders (Bonsit) was used to better the connection between successive parts. The beam was tested, on a 6-ft. span, with a center

load, the first application, up to 2 500 lb., being applied at the neutral axis on two projecting rods which were cast in the beam for the purpose. The concrete section was 8 by 4 in. over all, reinforced with two ½-in. smooth round rods placed ½ in. above the bottom of the beam, each one extending from one end to a point about 1 ft. from the other point of support. Each end was bent upward in a comparatively

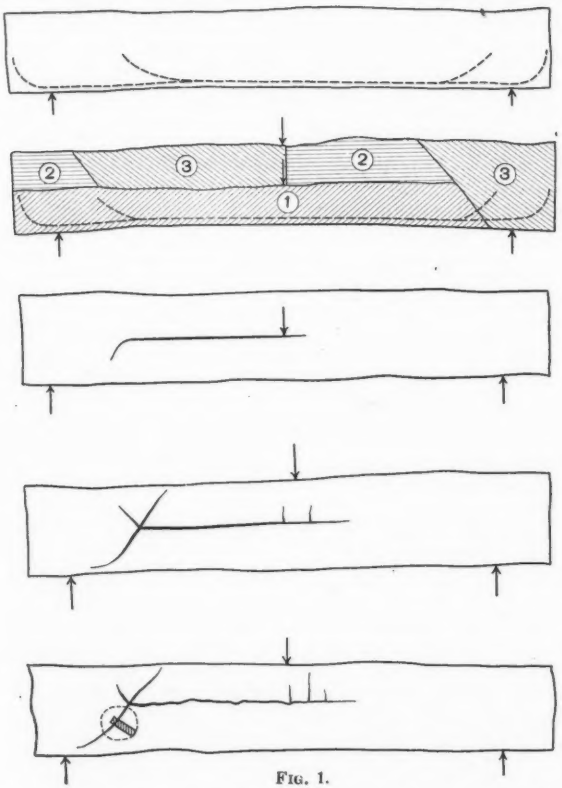

FIG. 1.

large-radius turn, the one at the end through an angle of 90° and the other one through about 45°, so as to serve to some extent in resisting diagonal tensile stresses. The reinforcement aggregated 1.34% of the effective beam area. The beam weighed about 180 lb. The load was applied in increments of 500 lb. each, the effect of each being carefully noted.

The shapes of the sections were determined with the following objects in view:

(*a*) To determine the effect of a horizontal joint in a beam, even when bonded as well as possible, where no steel is used for the purpose, when the load is applied at the level of the joint. This is the condition which exists in beams which are designed to be as deep as the distance between vertically adjacent window heads and sills in a concrete building, and where the concrete is almost invariably deposited in two separate operations, with the joint between them occurring at the level of the concrete floor.

(*b*) To ascertain whether it were possible to secure, along surfaces theoretically close to those on which maximum diagonal tensile stresses are developed, a bond of sufficient strength to make it unobjectionable to stop work along such a surface.

(*c*) To ascertain whether there is any virtue in leaving a joint only along a surface theoretically close to those on which maximum diagonal compressive stresses are developed.

The test results obtained were as follows:

(*a*) At 2 500 lb., applied at the center, a horizontal crack appeared, extending from the load toward one end and turning sharply downward into the mass concrete close to the turned-up end of one rod.

(*b*) The load was then shifted to the top of the beam, whereupon the horizontal crack disappeared entirely and the diagonal one was barely visible.

(*c*) At 1 500 lb. applied on the top, a condition almost identical with (*a*) had been reached.

(*d*) At 3 500 lb., the diagonal crack had extended downward and was following along the reinforcing rod toward the end of the beam. Very small tension cracks had appeared in the upper part of the beam under the load, and a wedge-shaped piece was commencing to crack out at a point where the diagonal and horizontal cracks intersected, in the vicinity of, but not along, an artificial joint parallel with diagonal compression planes.

(*e*) The load gradually fell to about 3 000 lb., as more strain was produced, and all these cracks became more prominent, until a sliver of concrete was forced off, revealing the end of one rod cutting directly across the main diagonal crack.

The rod had slipped in the concrete near the end of the beam, but adhered firmly to that on the other side of the crack.

It is to be noted that the artificial joint, where diagonal failure

was expected to occur, was closer to one point of support than was the crack which actually occurred at the other end. Thus the diagonal stresses were larger on the artificial joint than they were where the mass concrete failed.

The results obtained in this particular beam may be summarized as follows:

(*a*) In a beam, a horizontal joint not reinforced with steel, even if carefully bonded with acid and grout, is such a cause of weakness that the beam should be designed of a height only up to the joint, whenever the load is applied at the level of this joint.

(*b*) In a beam, a joint can be made sloping in the direction of the cracks caused by diagonal tensile stresses, which will be as strong as mass work, if the joint is bonded in accordance with the specifications given above.

(*c*) In a beam, the concrete in the vicinity of a diagonal joint parallel with the lines of maximum diagonal compressive stresses will fail before a joint made oppositely does, if incipient failure has already occurred in the vicinity of what may be called the compression joint and has not done so near the tension joint.

As a general conclusion, it may then be stated, that if proper bonding methods are adopted, with acid and grout, work may be stopped along 45° planes at any point in a beam, at pleasure, between the ends and the third points. In view of the one other test reported besides the writer's, such planes would better be diagonal compression planes. The writer prefers such joints to vertical ones at the centers of beams, where direct tensile and compressive stresses are larger. Further, it is to be remarked that nothing but ample reinforcement, consisting of stirrups or numerous diagonal rods, can compensate for the detrimental effect of a horizontal joint left along or near the central axis of a beam. The writer prefers numerous stirrups, well connected to the tension rods, and amply anchored in the top concrete. In proof of the efficacy of this design, reference may be made to his brick beam experiment, and the one in which no concrete at all was placed around the tension rods.*

* *Transactions*, Am. Soc. C. E., Vol. LVI, p. 343.

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PAPERS AND DISCUSSIONS

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STRESSES IN MASONRY DAMS.*

By William Cain, M. Am. Soc. C. E.

The object of this investigation is to determine the amounts and distribution of the stresses in a masonry dam, at points not too near the foundations, having assumed the usual "law of the trapezoid," that vertical unit pressures on horizontal planes vary uniformly from face to face.

Experiment indicates that such vertical stresses increase pretty regularly in going from the inner to the outer face, for reservoir full, until we near the down-stream or outer face, where the stress gradually changes to a decreasing one, which decrease continues to the end of the horizontal section. The law of the trapezoid is thus only approximately true over part of the section, but, as it gives an excess pressure where it attains a maximum, it errs on the safe side.

The profile of the dam selected is of the triangular type, with some additions at the top, but the method used in determining the stresses is general, and will apply to any type of profile. The final equations will give at any (interior or exterior) point of the horizontal section considered the vertical unit stress on the horizontal

* This paper will not be presented at any meeting, but written communications on the subject are invited for publication with it in *Transactions*.

section, the normal stress on a vertical plane, and the unit shear on either horizontal or vertical planes. From these stresses, the maximum and minimum normal stresses, and the planes on which they act, can be determined, and ultimately, if desired, the stress on any assumed plane can be ascertained.

The solution presented is approximate, which is justifiable, in view of the approximation involved in "the law of the trapezoid" used. The results, however, are practically correct, as will be evident from the checks applied, resulting from the exact theory given in the Appendix. The theory used, being simple, should be easily followed.

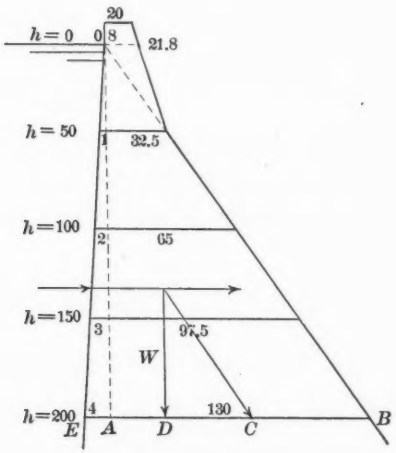

FIG. 1.

Let Fig. 1 represent a slice of the dam contained between two vertical parallel planes, 1 ft. apart and perpendicular to the faces. The batter of $O\,B$ is, $\frac{130}{200} = \frac{0.65}{1}$; that of $O\,E$ being $\frac{4}{200} = \frac{0.02}{1}$. The batter of the inner face was found by trial, so that the centers of pressure on horizontal sections, for reservoir empty, should nowhere pass more than a fraction of a foot outside the middle third of the section. The simple type of profile shown was adopted for ease of computation.

For convenience in subsequent computations, the breadth, $b = E\,B$, of horizontal sections, corresponding to various depths, h, below the

surface of the water in the reservoir, are given, all dimensions being in feet:

$$h = 199.0,\ b = 133.330$$
$$h = 199.5,\ b = 133.665$$
$$h = 200.0,\ b = 134.000$$
$$h = 200.5,\ b = 134.335$$
$$h = 201.0,\ b = 134.670$$

Take the weight of 1 cu. ft. of masonry equal to 1; then the weight of masonry above any section is equal to the corresponding area in Fig. 1 above that section. The area of the portion above *E O B* is readily found to be 712, and its moment about the vertical, *A O*, is 11 603, the unit of length being the foot. In Fig. 1, *D* is where the vertical through the center of gravity of the dam above the joint, *E B*, cuts that joint, and *C* is the center of pressure on that joint when the water pressure on *E O* is combined with the weight of masonry, *W*, above *E B*.

As *h* varies, suppose each horizontal joint, in turn, marked similarly to the joint at $h = 200$, with the letters *E, A, D, C, B;* then, for any joint, on taking moments of the triangles, *A O B, A O E,* and the area above *O B*, we find,

$$A\,D = \frac{\frac{A\,O}{6}\left(A\,B^2 - E\,A^2\right) + 11\,603}{W}$$

Assuming that the masonry weighs $2\frac{1}{2}$ times the water per cubic unit, then the weight of a cubic foot of water is $\frac{2}{5}$. It would entail but little extra trouble here, where the inner face has a uniform batter throughout, to include the vertical component of the water pressure on the face, *E O;* but it will be neglected, as usual.

The horizontal water pressure for the height, *h*, is thus, $\frac{2}{5} \times \frac{h^2}{2} = \frac{1}{5}\,h^2$, and its moment about *C* is, $\frac{1}{5}\,h^2 \times \frac{1}{3}\,h = \frac{1}{15}\,h^3$.

Taking moments of *W* and water pressure about *C*, we have at once,

$$D\,C = \frac{1}{15} \times \frac{h^3}{W}.$$

From the last two formulas, we derive the following results:

h	*W*	*A D*	*D C*
199	13 978.335	40.49141	37.58483
200	14 112.000	40.70316	37.79289
201	14 246.335	40.91488	38.00089

A seven-place logarithmic table was used throughout, the aim in the computations being to get the seventh significant figure correct within one or two units. The necessity for this accuracy will be seen later.

The distances, $E\,C$ and $C\,B$, are now readily derived.

For, $h = 199$, $E\,C = 82.05624$, $C\,B = 51.27376$

$h = 200$, $E\,C = 82.49605$, $C\,B = 51.50395$

$h = 201$, $E\,C = 82.93577$, $C\,B = 51.73423$.

On any plane, $E\,B$, the vertical unit pressure,

$$\text{at } B = p_1 = \frac{4\,b - 6\,C\,B}{b^2}\,W,$$

$$\text{at } E = p_2 = \frac{4\,b - 6\,E\,C}{b^2}\,W;$$

where $b = E\,B$, and W is the weight of masonry above the plane. This follows from the assumed "law of the trapezoid."

From these formulas we derive,

$$\text{At } h = 199,\ p_1 = 177.45483,\ p_2 = 32.22542,$$
$$h = 200,\ p_1 = 178.3855,\ p_2 = 32.24139,$$
$$h = 201,\ p_1 = 179.3160,\ p_2 = 32.25798.$$

Call p the vertical unit stress at a distance, x', from E; then,

$$p = p_2 + \frac{p_1 - p_2}{b}\,x';$$

and the total stress on the base, x', is,

$$P = \frac{1}{2}\,[p_2 + p]\,x' = p_2\,x' + \frac{p_1 - p_2}{2\,b}\,x'^2 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(1)$$

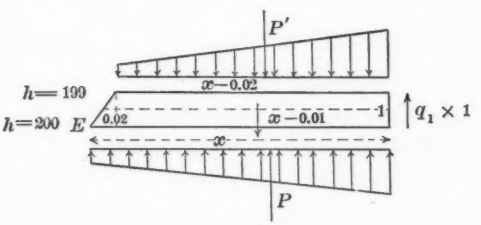

FIG. 2.

To find the unit shear on vertical or horizontal planes*, consider a slice of the dam, bounded by horizontal planes at $h = 199$ and

* The writer desires here to acknowledge his indebtedness to a recent paper on "Stresses in Masonry Dams" by Ernest Prescot Hill, M. Inst. C. E., published in *Minutes of Proceedings*, Inst. C. E., Vol. CLXXII, p. 134. Mr. Hill considers the case of a dam with a vertical inner face. By the aid of the calculus, he effects an exact solution, which leads to general formulas for shear and normal pressures on vertical planes.

Mr. Hill ascribes to Professor W. C. Unwin the suggestion, "that the shearing stress at any point may be found by considering the difference between the total net vertical reactions [between that point and either face] along two horizontal planes at unit distance apart," and states that Prof. Unwin "has applied the principle to a triangular dam by the use of algebraical methods."

$h = 200$, the water face and a vertical plane, at a distance, x, from the inner face (Fig. 2), in equilibrium under the water pressure acting horizontally on its left face and the forces exerted by the other parts of the dam on the slice. These forces consist of the uniformly increasing stress, P', on top, acting down; the uniformly increasing stress, P, on the bottom, acting up; a shear acting on the vertical plane at the right, of average intensity q_1 per square foot; the weight of the body $(x - 0.01)$, besides the horizontal forces to be given later. The vertical component of the water pressure is here neglected, as usual. The origin for x is taken, here and in all subsequent work, at the level, $h = 200$, at the inner face.

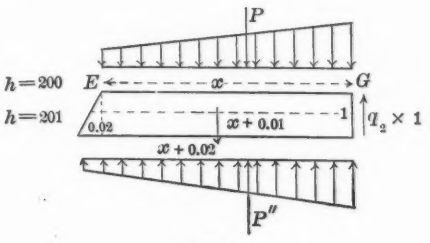

FIG. 3.

For equilibrium, the sum of the vertical components must be zero. Therefore, $q_1 = (x - 0.01) + P' - P$ .....(2)

To find P', substitute in Equation 1, $x' = x - 0.02$, $p_2 = 32.22542$, $p_1 - p_2 = 145.22941$, $b = 133.330$, giving $P' = 32.20364\ x + 0.5446238\ x^2 - 0.6442906$. P, $x' = x$, $p_2 = 32.24139$, $p_1 - p_2 = 146.1441$, and $b = 134$; therefore

$$P = 32.24139\ x + 0.5453138\ x^2.$$

Substituting in Equation 2, we derive the average unit shear,

$$q_1 = -0.6542906 - 0.96225\ x - 0.0006900\ x^2 \text{ ..(3)}$$

This value of q_1 is strictly correct when $x \geqq 0.02$. It is slightly in error when $0 < x < 0.02$.

A similar investigation holds to obtain the average unit shear q_2 (Fig. 3) on a vertical plane, at a distance, x, from E, extending from the level, $h = 200$, to the level, $h = 201$.

We have, for equilibrium,

$$q_2 = (x + 0.01) + P - P'' \text{ .....(4)}$$

We find P'' by substituting in Equation 1, $x' = (x + 0.02)$, $p_2 = 32.25798$, $p_1 - p_2 = 147.05802$, and $b = 134.67$. $P'' = 32.27982\ x +$

$0.5459941\,x^2 + 0.6453780$. Substituting this, and the value previously found for P, in Equation 4, we derive,

$$q_2 = -0.6353780 - 0.96157\,x - 0.0006803\,x^2 \ldots\ldots (5)$$

This is strictly correct only when $x \geqq 0$.

The mean, $\frac{1}{2}(q_1 + q_2)$, of these average shears will be assumed as approximately equal to the intensity of shear at the point, G ($x = E\,G$), at the level, $h = 200$. Call q this intensity of shear on a vertical plane at G; therefore,

$$q = -0.6448343 + 0.96191\,x - 0.0006856\,x^2 \ldots\ldots (6)$$

Checks.—By Appendix (b) and (d), the exact value of q, at either face, $= p$ tan. ϕ, where p = vertical unit normal stress at the face and ϕ is the angle the face makes with the vertical. Thus, at the inner face, $q = -32.24139 \times 0.02 = -0.6448278$, whereas Equation 6 gives, for $x = 0$, $q = -0.6448343$.

At the outer face, the exact value is, $178.3855 \times 0.65 = 115.9506$, whereas Equation 6 gives, for $x = 134$, $q = 115.9405$.

A still more searching test can be devised. It is a well-known principle that the intensity of shear at a point, on vertical or horizontal planes, is the same [Appendix (a)]. Therefore, regarding Equation 6 as giving the horizontal unit shear, at the level, $h = 200$, where $b = 134$ ft.; the total shear, from face to face, on this level, is,

$$\int_{x=o}^{x=134} q\,dx = 7999.75$$

This should equal the total water pressure down to the same level, $\frac{1}{5}(200)^2 = 8\,000$. Formula 6 thus gives practically exact results.

In order to find the normal unit stress on a vertical plane, we shall assume that q_1, given by Equation 3, equals the intensity of shear on a vertical or horizontal plane at the point x, at $h = 199.5$; and that q_2, given by Equation 5, gives the shear intensity at x at $h = 200.5$. This evidently supposes that the shear intensity increases uniformly, vertically, from $h = 199$ to $h = 201$.

Consider a portion of the dam, Fig. 4, bounded by the water face; the plane, $F\,M$, at the level, $h = 199.5$, on which the total shear is Q', the plane, $E\,N$, at the level 200.5, on which the total shear is Q, and the vertical plane, $M\,N$, 1 sq. ft. in area, on which the average normal stress is p'. The water pressure on $E\,F$ will be supposed to be exerted horizontally. It is equal to 80 units. Assuming, as stated, that $q_1 =$

intensity of horizontal shear at M, and q_2 = the corresponding intensity at N, we have, taking the origin as before O,

$$Q' = \int_{0.01}^{x} q_1 \, dx; \qquad Q = \int_{0.01}^{x} q_2 \, dx;$$

or,

$$Q' = 0.006494794 - 0.6542906\, x + 0.481125\, x^2 - 0.00023\, x^3$$

$$Q = -0.00640186 - 0.6353780\, x + 0.480785\, x^2 - 0.0006803\, \frac{x^3}{3}$$

Checks.—The total water pressure for $h = 199.5$ is $\frac{1}{5}(199.5)^2 = 7960.05$ and for $h = 200.5$, $\frac{1}{5}(200.5)^2 = 8\,040.05$. The first should equal Q', for $x = 133.665$, or 7 959.22; the second should equal Q, for $x = 134.335$, or 8 041.12. The slight differences tend to give confidence in the results.

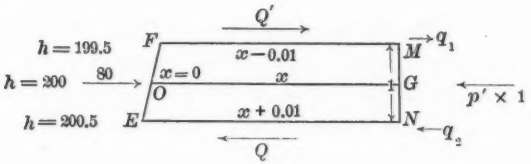

FIG. 4.

For equilibrium, the sum of the horizontal forces acting on $E\ F\ M\ N$, Fig. 4, must be zero;

$$\text{therefore, } p' = 80 + Q - Q' \ldots\ldots\ldots\ldots\ldots\ldots (7)$$

$$p' = 80.01 - 0.0189\, x + 0.00034\, x^2 - 0.00000323\, x^3.$$

This average stress will now be assumed to be the intensity of the horizontal unit stress on vertical planes at $h = 200$.

It will now be perceived why a seven-place table was necessary in the computations, the coefficients of x^2 and x^3 having only two or three significant figures in the final result. If the planes originally had been taken 0.1 ft. apart vertically, a ten-place table would have been required.

Checks.—The value of p', for $x = 0$, $p' = 80.012896$, is the same as that given by Appendix (a), $80 + 0.6448 \times 0.02$. When $x = 134$, the formula gives $p' = 75.81$, whereas the exact theory, Appendix (b), gives $p' = m^2\, p = (0.65)^2 \times 178.39 = 75.37$. The difference is 0.44 at the outer face. For any other point, it might be assumed to vary with x, so that it could be corrected by subtracting $\frac{0.44}{134} x = 0.0033\, x$

from the value of p' above. For ease of computation, the formula will be written,

$$p' = 80.01 - 0.02\,x + 0.00034\,x^2 - 0.00001\,\frac{x^3}{3} \quad \ldots\ldots(8)$$

The first coefficient of x^3 cannot be counted on to the last two figures, hence we are permitted to change 323 to 333 in that coefficient. When $x = 134$, Equation 8 gives $p' = 75.41$, nearly the exact value.

The three formulas for p, q and p', at the level $h = 200$, are thus as follows:

$$p = 32.24 + 1.09063\,x.$$

$$q = -0.64 + 0.962\,x - 0.000686\,x^2.$$

$$p' = 80.01 - 0.02\,x + 0.00034\,x^2 - 0.00001\,\frac{x^3}{3}.$$

Since the weight per cubic foot of masonry was assumed as two and one-half times that of water, we must multiply the stresses given in Table 1 by $\frac{5}{2}\,(62.5) = 156.25$, to reduce to pounds per square foot; or by 1.085, to reduce to pounds per square inch.

TABLE 1.

x	0	10	25	50	75	100	134
p	32.24	43.15	59.50	86.77	114.04	141.30	178.39
q	−0.64	8.91	22.98	45.75	67.65	88.70	115.95
p'	80.01	80.02	79.66	79 11	79.01	78.08	75.37
max. f	80.02	82.06	94.67	128.85	166.40	208.85	253.71
min. f	32.23	41.11	44.48	37.03	26.64	15.52	0
θ for max. f	90°46′	77°06′	56°50′	42°36′	37°44′	35°12′	33°01′

In Table 1 the stresses are those experienced at the level, $h = 200$.

p = vertical unit stress on a horizontal plane,

q = shearing unit stress on horizontal or vertical planes,

p' = horizontal unit stress on vertical planes,

Max. f = maximum normal stress acting on a plane inclined to the horizontal at the angle θ, given on the last line,

Min. f = minimum normal stress acting on a plane perpendicular to the last.

From max. f and min. f, with θ, the ellipse of stress can be drawn, and the stress in any direction, with the plane on which it acts, can be ascertained.

It will be observed that there is no tension exerted anywhere, and that the maximum compression is 253.71, or 275 lb. per sq. in., which

is exerted at the outer face, parallel to that face, upon a plane at right angles to the face.

In Appendix (*e*), the important formula, for the maximum normal intensity at the outer face, acting parallel to that face,

$$f = \frac{p}{\cos.^2\phi}$$

is proved. In this instance, $p = 178.39$, tan. $\phi = 0.65$, therefore $\phi = 33°01'$, whence $f = 253.71$.

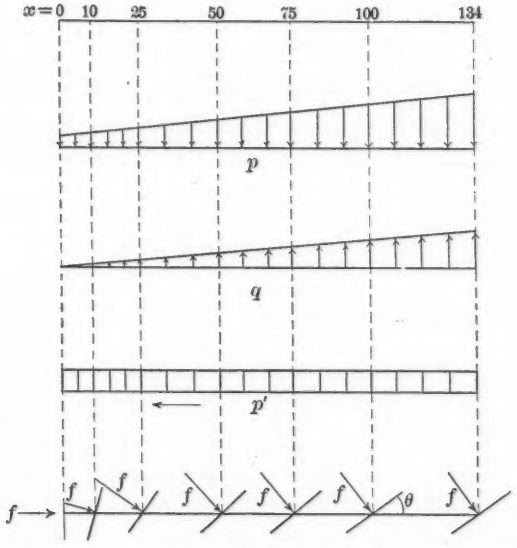

FIG. 5.

This stress is unaccompanied with any conjugate stress, perpendicular to the face. In the interior of the dam, where conjugate stresses prevail, the masonry is perhaps better able to withstand a certain compressive stress than at the face. The distribution of stresses, at the level $h = 200$, is shown in Fig. 5, on the supposition that the base of the dam is a little below that level. The connection with the foundation materially modifies this distribution; but Fig. 5 shows the distribution for sections, say, from 10 to 20 ft. above the base, up to the level $h = 50$, fairly well, on the basis of the trapezoid law. As has been mentioned before, this law gives a pressure greater than the actual at the outer face.

Since the batter of the inner face is very small, the results of Table 1 should agree approximately, except near the inner face, with those found by Mr. Hill in the paper referred to in the foot-note. Substituting numerical values, Mr. Hill's formulas, for $h = 200$, reduce to,

$$q = 0.9426\ x - 0.0005768\ x^2,$$
$$p' = 80 - 0.0001289\ x^2 - 0.0000009615\ x^3;$$

giving,

x	0	10	25	50	75	100	134
q	0	9.36	23.20	45.69	67.45	88.49	115.95
p'	80	79.99	79.90	79.56	78.87	77.75	75.38

On comparing these formulas with those of the writer, it will be observed that the absolute term in the value of q and a consequent term of the first degree in x, in the value of p', are lacking in Mr.

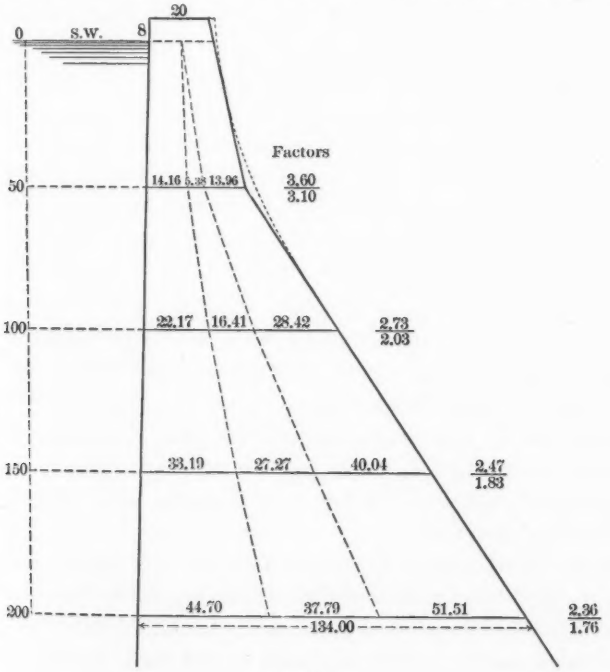

FIG. 6.

Hill's formulas. This results from taking the inner face as vertical. Although the coefficients also differ, it is seen that the numerical values are very nearly the same.

In Fig. 6 are shown, on a drawing of the dam, to scale, the lines of the centers of pressure for reservoir full and empty.

To the right, and under the word "factors," are certain numbers, written in the form of fractions. For any joint, the upper number gives the factor against overturning, or the number by which it is necessary to multiply the water pressure down to the joint, to cause the total resultant to pass through the outer edge of the joint considered. The lower numbers give the ratio of the weight of masonry above a joint to the water pressure corresponding.

It is believed that these "factors" should increase from the base upward, to allow somewhat for earthquakes, expansion of ice in freezing, etc., since the effect of such accidental forces is proportionately greater on the upper joints.

Stresses due to water infiltration are not included here; neither are stresses due to temperature changes.

The unit stresses, f, in pounds per square inch, acting parallel to the adjacent face, are as follows, and refer to the outer edges of the joints, for reservoir full, and to the inner edges, for reservoir empty:

h,	f at outer edge,	at inner edge.
50	85	58
100	136	133
150	204	180
200	275	228

The stresses, f, are normal pressures on planes perpendicular to the respective faces, and are the greatest stresses that can be experienced in the dam. In fact, they are greater than the true stresses, since the trapezoid law is not exact, particularly near the base, as before remarked. It would then seem that the dam, thus far, is safe, since the maximum unit stress is less than concrete, even, is subjected to daily, in good practice.

For an actual construction, the outer face should be curved, from near $h = 50$ to the top, as shown by the curved dotted line in Fig. 6.

The subject of the stresses in masonry dams has caused a great deal of discussion among British engineers in the last two or three years. The subject was reopened by Mr. L. W. Atcherly and Professor Karl

Pearson,* who gave the results of certain experiments which seemed to indicate considerable tension across vertical planes near the outer toe. The late Sir Benjamin Baker, Hon. M. Am. Soc. C. E., also published† the results of experiments on a model dam of stiff jelly, and very recently, the "Experimental Investigations" of Sir J. W. Ottley and Mr. A. W. Brightmore‡ on elastic dams of "plasticine" (a kind of modeling clay) and the experiments of Messrs. J. S. Wilson and W. Gore§ on "India Rubber Models" have been presented.

It is not the object of this paper to discuss these later experiments; but it may be remarked that they show very plainly that no tension exists near the outer toe, but that tension does exist at the inner toe, where the dam is joined to the foundation, and it has become a serious matter how to deal with it. The influence of the foundation in modifying the distribution of the stresses at the base of the dam was found to be very great, causing the shear there to be more uniform than higher up, where the parabolic law, nearly as given by the formulas above, was found to hold. Also, above some undetermined plane, a small distance above the base, the usual "law of the trapezoid" was found to be approximately correct, leading to stresses on the safe side at the outer toe. This law leads to stresses at the outer toe of the base considerably in excess of the true ones.

It was found, from the rubber models particularly, as theory indicates, that the greatest normal pressures are exerted at the downstream face, for reservoir full, and they act in a direction parallel to that face.

* *Minutes of Proceedings*, Inst. C. E., Vol. CLXII, p. 456.
† *Ibid*, Vol. CLXII, p. 123.
‡ *Ibid*, Vol. CLXII, p. 89.
§ *Ibid*, Vol. CLXII, p. 107.

APPENDIX.

Relations Between Stresses at any Point of a Dam.

(*a*).—Consider a cube of masonry, Fig. 7, the edge of which has the length a, bounded by vertical and horizontal planes and subjected to normal and shearing forces, caused by the action of the other parts of the dam. Since a will be supposed to diminish indefinitely, the weight of the cube, which is proportional to a^3, is an infinitesimal of the third order, and can be neglected in comparison with the normal forces, which vary as a^2 and are thus of the second order.

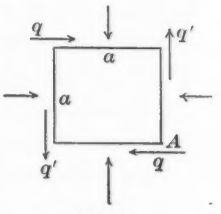

Fig. 7.

Similarly, the average unit stresses exerted on the faces can be treated from the first as the unit stresses at any point, A, of the cube. As a diminishes indefinitely, the oppositely directed normal forces approach equality and balance independently; hence the couples formed by the shears on opposite faces must likewise approach equality; the one being right-handed, the other left-handed; therefore $q\ a \times a = q'\ a \times a$, or $q = q'$; hence, the intensities of shear at a point on two planes at right angles are equal. The relative directions of the shears on two planes at right angles are determined, as above, from the consideration that one resulting couple must be right-handed and the other left-handed. This applies also to Figs. 8 to 11.

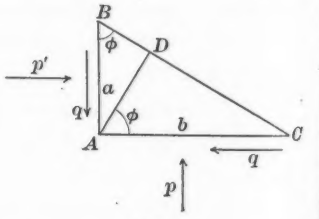

Fig. 8.

(*b*).—In Fig. 8, $A\ B\ C$ is the right section of a prism at the outer face, with lateral faces one unit in length, perpendicular to the plane of the paper.

Let $A\ B$ be vertical; tan. $\phi = m$, a constant;

p = normal intensity on a horizontal plane at C,

p' = normal intensity on a vertical plane at C,

q = shear intensity on horizontal or vertical planes at C.

The weight of the prism is, $\frac{1}{2}\ a\ b$.

Balancing vertical as well as horizontal components, we have, when $a = A\ B$ and $b = A\ C$ are very small,

$$p\ \ b = q\ a + \frac{1}{2}\ a\ b, \text{ nearly;}$$
$$p'\ a = q\ b.$$

Dividing the first equation by b, the second by a, the limit, as a and b approach zero, gives exactly,

$$p\ = q \text{ cot. } \phi, \text{ therefore } q = m\ p,$$
$$p' = q \text{ tan. } \phi, \text{ therefore } p' = m^2\ p,\ p\ p' = q^2.$$

These equations give the relations between p, q and p' at the outer face. The same relations hold at the inner face, for reservoir empty, on replacing ϕ by ϕ', the angle the inner face makes with the vertical.

For the remaining cases, the final limits will be written at once, since the complete process of deriving them is evident from the above. In fact, the weight of the prism, $\frac{1}{2}\ a\ b$, being of the second order, can be neglected in comparison, with $q\ a$, etc.

(c).—For reservoir full, calling $w = \frac{2}{5}\ h$, the intensity of water pressure, horizontally or vertically, at C, we have at the inner face, putting tan. $\phi' = n$, Fig. 9,

$$p\ b = q\ a + w\ b;\ p'\ a = q\ b + w\ a;$$

therefore $\quad p = \frac{1}{n}\ q + w;\ p' = q\ n + w.$

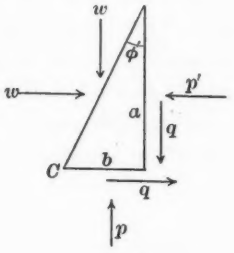

FIG. 9.

(d).—If the vertical component of the water pressure is neglected, these equations reduce to,

$$p = \frac{1}{n}\ q;\ p' = q\ n + w;$$

therefore $\quad q = p\ n,\ p' = n^2\ p + w.$

(e).—Since the shear on the outer face is zero, therefore, by (a), the shear on a plane, $A\ D$, Fig. 10, perpendicular to the outer face, is also zero, or the stress on $A\ D$ is normal.

Call f the intensity of such a stress at C. The total pressure on $A\ D = f \times A\ D = f\ b$ cos. ϕ, and its vertical component is $f\ b$ cos.$^2\ \phi$, therefore balancing the vertical components,

$$p\ b = f\ b \cos.^2\phi;$$

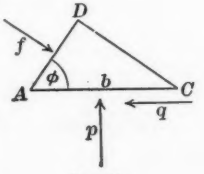

FIG. 10.

therefore $$I = \frac{p}{\cos.^2 \phi} = p \sec.^2 \phi$$

This is a most important formula for finding the maximum normal intensity at the outer face. It applies equally to the inner face for reservoir empty, on changing ϕ to ϕ', the angle the inner face makes with the vertical. For either face, p is the vertical normal unit stress at the face considered.

(*f*).—*Principal Normal Stresses at Any Point in the Dam and the Planes on Which They Act.*—In the prism, *A B C*, Fig. 11, let *A B* be one of the planes on which the stress is normal. Let f be its intensity. The stress on the plane, *A B*, of unit length perpendicular to the plane of the paper, is thus $f\ c$; its vertical component is, $f\ c$ cos. $\theta = f\ b$, and its horizontal component is $f\ c$ sin. $\theta = f\ a$, θ being the angle that *A B* makes with the horizontal.

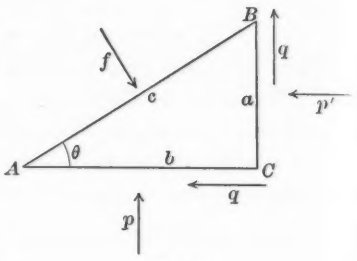

FIG. 11.

Place the sum of the vertical forces acting on *A B C* equal to zero; also place the sum of horizontal forces equal to zero.

$$f\ b = p\ b + q\ a, \text{ therefore } f - p = q \text{ tan. } \theta,$$
$$f\ a = q\ b + p'\ a, \text{ therefore } f - p' = q \text{ cot. } \theta.$$

The difference of the last two equations gives,

$$p - p' = q \text{ (cot. } \theta - \text{tan. } \theta) = q \frac{1 - \text{tan.}^2\ \theta}{\text{tan. } \theta},$$

therefore $$\text{tan. } 2\ \theta = \frac{2 \text{ tan. } \theta}{1 - \text{tan.}^2\ \theta} = \frac{2q}{p - p'}.$$

The angles, θ (differing by 90°), computed from this equation, give the directions of the planes, *A B*, on which the stress is entirely normal.

From an equation above, we likewise have,

$$\text{tan. } \theta = \frac{f - p}{q}.$$

This gives directly the plane on which a given f acts.

To deduce a formula for f, take the product of two equations above.

$$(f - p)\ (f - p') = q^2,$$

therefore $$f = \frac{1}{2}\ [p + p' \pm \sqrt{(p + p')^2 - 4\ (p\ p' - q^2)}].$$

This equation gives the two values of f corresponding to the two planes mentioned; compressive when f is positive, tensile when negative. There can be no tension when $p\,p' \geqq q^2$.

A better form for computation is,

$$r = \frac{1}{2}\,[p + p' \pm \sqrt{(p - p')^2 + 4q^2}].$$

Vol. XXXV. JANUARY, 1909. No. 1.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

THE IMPROVEMENT OF THE OHIO RIVER.

Discussion.*

By Theron M. Ripley, Assoc. M. Am. Soc. C. E.

Mr. Ripley. Theron M. Ripley, Assoc. M. Am. Soc. C. E. (by letter).—A reading of Major Sibert's paper brings again the query which has come to mind many times in the past few years, viz., on what data and after how careful consideration of the questions has been based the assumption that a possible 11 ft. is the maximum which should be provided for on the Ohio improvement, and if 9 ft. is economically necessary at Pittsburg at present, would not 11 ft. or more be the economical development below Portsmouth or Cincinnati?

This query is not an insinuation as to the paucity of data or lack of study of the scheme for the Ohio River as a river, but in its relation to the possibilities and probabilities of contiguous improvements and their bearing on the Ohio River work.

The State of Ohio contains at least one, and maybe two, routes along which it is possible to construct a canal from Lake Erie to the Ohio River with a depth of water of not less than 12 ft.

For several years a determined effort has been made (and is now being made) by some of the State's best men to have such a canal constructed. There are those who believe that the State could expend no money which would be of greater economic benefit than in building such a canal.

Ohio is geographically situated directly between the immense ore deposits of Northern Michigan and the no less immense deposits of coal

* This discussion (of the paper by William L. Sibert, M. Am. Soc. C. E., printed in *Proceedings* for October, 1908) is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

in West Virginia. Already traversed by the trunk lines of nearly all the railroads from the Atlantic to St. Louis and northern points, what more natural than that some of her citizens should believe in bringing this coal and iron together by the cheapest method, and shipping her manufactured product by her own waterway and the Barge Canal of New York State to New York City, or by her railroads to Atlantic and inland points farther south, or by her canal and river to New Orleans and intermediate points? Mr. Ripley.

Questions such as these should be taken into consideration in any development for the Ohio, as any structures in that river will determine the economic navigable depth of connecting waterways above, and may assist or destroy their usefulness. In fact, a less depth below Portsmouth or Cincinnati than that possible across the State of Ohio might prevent the building of an Ohio Canal, and in any event would be a serious handicap thereto.

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PAPERS AND DISCUSSIONS

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ELECTRIC RAILWAYS IN THE OHIO VALLEY BETWEEN STEUBENVILLE, OHIO, AND VANPORT, PENNSYLVANIA.

Discussion.*

BY MESSRS. F. LAVIS, GEORGE B. PRESTON, J. MARTIN SCHREIBER, AND WILLIAM J. BOUCHER.

Mr. Lavis.

F. LAVIS, M. AM. SOC. C. E.—The valley of the Ohio River and the territory tributary to it have witnessed a greater development of long-distance interurban electric railroads than any other part of the United States. Many of these lines run through sleeping, dining and drawing-room cars, built by the Pullman Company, of only slightly lighter construction than the standard equipment of the largest trunk lines of steam railroads. A description, therefore, of one of these lines, completed only about six months ago by one of the most prominent engineering firms identified with work of this kind, is presumably a description of the very latest and best practice in the construction of railroads of this type at the present time.

The general characteristics of these interurban lines are, that they pass through the various cities and towns along the route on the surface of the streets, and as near the business centers as possible, while in the open country between, they are located, for the greater part of the distance, on private right of way, in many instances having numerous highway crossings at grade. This latter feature is not mentioned by the author, and may have been avoided on this road, but the

* This discussion (of the paper by George B. Francis, M. Am. Soc. C. E., printed in *Proceedings* for November, 1908), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

number of these crossings, which are only slightly, if at all, less dangerous than those of steam railroads, is becoming increasingly large, even in States where a great deal of public money has been, and is being, expended in eliminating such features on steam railroad lines. Mr. Lavis.

This road, between Rochester and Steubenville, passes through a territory described by the author as a very "hive of industry," and the population tributary to it, amounting to more than a million people, or practically 100 000 per mile on the whole length between Pittsburg and Wheeling, is composed largely of the class of prosperous mechanics and laborers which furnishes a larger patronage, in proportion to the total population, to a road of this kind than does any other. The most substantial construction, therefore, is apparently warranted.

In order to get some idea of the relative importance of the construction on such a line, it seems desirable to have some basis of comparison with other standard railroad practice. The bridges on this line are designed for a concentrated load of 24 tons on two axles at 10-ft. centers, or 1 800 lb. per lin. ft. of single track. The heaviest equipment of the New York Subways and Elevated Lines has a load of 30 tons on the two axles with the motors, and 11 tons on the other two, or an average of 2 080 lb. per lin. ft. of track, which is only slightly greater than that used on this line. The largest Pullman cars in use on steam railroads are about 80 ft. long, and weigh about 60 tons, this load being carried on two six-wheeled trucks, giving about 10 tons per axle and about 1 500 lb. per lin. ft. of track. A passenger locomotive of the Atlantic type, weighing about 75 tons, which is a fairly large engine for express passenger service on anything but mountain grades, has a load of about 20 tons per axle on the driving wheels, and the load, for engine and tender loaded, will be about 4 000 lb. per lin. ft. of track. A consolidation freight engine, weighing about 100 tons, has a load of about 22 tons on the driving axles, and will average 5 000 lb. per lin. ft. of track. The bridges on this road, therefore, will carry almost any standard railroad equipment except locomotives, the weights of the heavier types of which in ordinary use would be about double that for which these bridges were designed.

Having in view, therefore, the fact that the requirements of roads of this class are not, in many respects, very far removed from standard steam railroad practice on first-class lines, and are in some respects greater than the requirements on many inferior lines, it has seemed to the speaker that it would greatly enhance the value of the paper if the author would give some of the reasons for the very radical departures from steam railroad practice as regards the location of this line.

The author states that "It required considerable engineering study and skill * * * to locate a line * * * which would give easy curves and grades," considering the natural difficulties of topography,

Mr. Lavis. and the possibility of securing private right of way. Nothing is stated in the paper in regard to what the author considers to be easy curves for a road of this type, although, as a matter of fact, outside of the cities, curves of from 200 to 300 ft. radius, provided there were not too many of them, and the amount of central angle covered was necessary, would probably have little effect either on cost of operation or speed. In passing through cities, curves of as small a radius as 50 ft. are not infrequent. This difference between roads of this type and trunk-line railroads where a curve of 1 000 ft. radius is considered quite sharp and necessitates slowing down to from 20 to 25 miles an hour, is quite marked, and is due of course to the absence on the electric road of the long rigid wheel base of the steam locomotive.

The question of grades on these electric roads, however, is one about which far less is known, at any rate publicly. Railroads building new lines to be operated by steam, even in a great deal of the, as yet, undeveloped parts of the country, are very reluctant to adopt higher rates than 0.6% unless the topography of the country absolutely demands it, whereas on the profile of the road under consideration, as shown by Fig. 2, one grade of 9.9% is shown for a fairly long stretch, and there are several stretches of 6% to 7%, although most of them seem to be fairly short.

The reason, of course, for the badly broken grade line is the fact that the river bank is occupied by the steam railroad, thus forcing the electric road in many cases high up on the side-hills where these come close to the river and necessitating its descent nearly to the river level where the hills lie farther back, in order to avoid a long detour to keep up. This characteristic of the country is shown very clearly in the view of the Yellow Creek Bridge.

The ability to ignore, to a very large extent, considerations which affect the determination of the rates of grade, which govern steam railroad practice, is the chief characteristic which differentiates the location of interurban electric railroads, or any electrically operated railroad where single cars or multiple-unit trains are to be used exclusively, from that of those operated by steam or electric locomotives at the head of the train, although few data have as yet been made public as to the actual conditions which govern the determination of the most economical grades to be used on these electric roads.

On a steam railroad, in conjunction with the necessity of low grades, it is also almost as important to arrange the grade line so that the demand on the locomotive will be as nearly even as possible throughout comparatively long stretches. No considerations of this kind apparently influenced the fixing of the grades on this railroad, and here again it would undoubtedly be of considerable interest to know just what considerations governed the judgment of the engineers in adopting the grades they did on this particular road.

Mr. Lavis.

As the steam locomotive is most economically operated when the demand on the machine is fairly consistent, so also is this the case with almost any machine or power plant, including, and perhaps especially so, electrical power plants.

In the case of the latter, when power is being manufactured for the operation of a railroad, it may be considered that, where a fairly large number of cars are being operated, the demand will be equalized by the fact that some cars will be going down grade while others are going up, some stopped while others are going at full speed, etc.; but, on a road such as the author describes, where the service is infrequent and the number of cars in use at a time small, the very broken grade line and the high rates of grade would apparently tend to create a somewhat irregular demand, as there would be nothing to prevent a combination frequently occurring where all the cars, or at least a majority of them, would be going up hill at once, and then within a few minutes be all going down hill. In the case of power-houses supplying power only for the operation of the cars on a railroad of this kind, this might be a serious consideration. The power-houses on this line, however, supply power for both manufacturing purposes and electric lighting, for which purposes the load is probably fairly even, and is a large proportion of the total output. The variation in the load from the operation of the cars, therefore, probably does not affect them greatly. The great economy of a uniform load at a power-station is strikingly exemplified by the contract recently made by the Commonwealth Edison Company, of Chicago, for supplying power to the street railway companies of that city.* The very low rate of 0.4 cent per kw-hr. was made, provided the demand was uniform, any variation from this uniform demand being penalized according to a certain schedule mutually agreed upon. Of course, a street railway operating in any large city will have high peak loads during the rush hours, and therefore the demand cannot be regular; this instance, however, being so recent, and the contract so important, serves to emphasize the desirability of uniform loads at the power-houses.

It is well to bear this in mind and not lose sight of the fact that the electrical operation of railways has its limitations, although many liberties may be taken with the grades and alignment that would not be permissible on steam railroads.

There is another question which it has seemed to the speaker might naturally occur to some of the readers of the paper, and that is: Before deciding on the construction of these lines at all, was any consideration given to the possible effect on the new line, should the steam railroad, with its truly easy grades and curves, decide to electrify its line and run short trains at frequent intervals, or even equip its line with one of the many types of motor cars now apparently in successful operation on many steam railroad lines, where frequent

* *Engineering News*, December 10th, 1908.

Mr. Lavis. service for a comparatively small number of passengers at a time is demanded? Probably the steam railroad would be at a disadvantage by its inability to reach the business centers of the various towns, and also to leave every passenger nearly at his own front door. It is quite probable, also, that such a service as that necessary to compete successfully with the electric lines might interfere too much with the through business of the road, both passenger and freight.

In constructing an interurban railroad across country, some years ago, where 60-ft. rails were used, the speaker had some trouble from the excessive expansion and contraction in such comparatively long lengths, until the ballast was filled in as close as possible to the top of the rails and on both sides of them. If the temperature changes are at all large, and the joints are left open sufficiently to prevent the track from kinking in hot weather, there is likely to be excessive pounding of the ends of the rails at the joints, due to the wide spacing. It would be interesting to know whether any such trouble has been found on this road; and, if so, what means were taken to overcome it. The standard cross-section of the track, shown in Fig. 3, seems to show that no ballast was placed above the tops of the ties.

It is quite common practice now, in laying street railway tracks through paved streets, to lay a fairly substantial concrete foundation both under and around the ties, and to such a height over them as will just allow for the type of paving to be used, with the necessary sand cushion under it. The speaker had occasion to note only a short time ago the installation of this type of construction for a street railway track operated by a large public service corporation, on a double-track line, where the cars were operated on 20 min. headway, with little if any special excursion business, so that the traffic was probably not greater than on the line described by Mr. Francis. No mention of such a type of track construction is made in the paper. If it was not used through the paved streets in the cities, was it on account of the necessity of curtailing expenses, or because it was not considered economical, in view of the expected traffic on the road? The excessive wear and tear on the motors of electric cars due to poor track has led to the very general adoption of the very best type of track construction possible for street railroads almost everywhere.

The width between centers of tracks, adopted on this road, namely, 9 ft. 8½ in., is the general practice on street railway work in cities, though many interurban roads, where high speed is used, have adopted a wider spacing through the open country. On the road recently completed between Baltimore and Washington the tracks were laid 11 ft. on centers, and 80-lb. rails in 33-ft. lengths were used. The fact that the bridges on the Ohio road were designed to allow the tracks to be spaced 12 ft. on centers allows the inference that the engineers had in view the possible necessity of wider spacing of the tracks in the future.

Mr. Lavis.

There is another point which the speaker believes is not made quite clear in the paper, and that is the reason for building a double-track road. The only information as to the amount of traffic is the statement that cars are operated on 30 min. headway for ordinary traffic, and that there is a considerable excursion business. On roads with cars operating normally on 30 min. headway, it is generally quite satisfactory to build a single-track road with sufficient sidings to provide for operation on half that headway, the speaker having in mind a road in New England which handles quite successfully a very large summer excursion business on this basis.

The construction of steam railroads is now carried out along lines which have become fairly well standardized, and the type of construction necessary to meet certain conditions is in a general way well known to engineers at all familiar with this branch of the profession. The construction of electric railroads, however, owing to the much greater flexibility of the application of power, is subject to much greater variations, and a most intimate co-relation of the work of the engineers responsible for the location of the line and the electrical engineers is necessary in order to produce the most economical results. The practice in the construction of this road, as described by Mr. Francis, is so radically different from steam railroad practice that the speaker believes that a fuller statement of the reasons which led to the final design, so to speak, of this road, would be most interesting, not only to the members of this Society, but to all engineers interested in work of this class.

Mr. Preston.

GEORGE B. PRESTON, ESQ.—As having some bearing on the relation between the average and maximum loads on the power stations, it may be of interest to note that with the heavy holiday load of July 4th, 1908, the 1-hour maximum at the East Liverpool house was 2 000 kw., while the average load during the heaviest part of the day, from 7 to 11 P. M., was 1 800 kw. On an ordinary day the 24-hour average for this station is 850 kw.

When considering the question of peak loads, in connection with this system, it should be borne in mind that these two power-houses are furnishing all the commercial and city lighting, as well as carrying a considerable commercial motor load, for four cities having an aggregate population of 73 000.

Mr. Boucher has asked whether or not the possibility of using the third-rail was considered in connection with this railway system. It was not considered feasible to use the third-rail system because of the necessity of utilizing the local electric railways at Steubenville and East Liverpool which were already installed and equipped with the overhead trolley, and also from the fact that a considerable portion of the roadbed lies in public streets and highways.

Mr. Schreiber.

J. MARTIN SCHREIBER, ASSOC. M. AM. SOC. C. E. (by letter).—Although no new engineering features, other than represented in modern practice, are brought out in the paper, it is an intelligent and comprehensive description of a very interesting electric railway.

It is gratifying, at least to those who have had experience in the maintenance of some of the old-time electric railways, to know that the capitalist is rapidly realizing that it even pays to spend some money on the permanent improvement of street railways. Indeed, the very near future will see almost all electric roads constructed with as much skill and forethought as is represented on steam roads. This will be necessary for economical, satisfactory, and profitable operation, and in order to compete with other carrying companies.

The physical difficulties encountered in building the track and roadway of the Ohio Valley lines were out of the ordinary, as the location was not favorable for the work, and the author seems to be justified in the assertion that the configuration of the country through which the road was built is such as practically to preclude the location and construction of a future competing line. No doubt this condition accounts for the variable and at some places severe grades of the track. However, since the road is double tracked, successful and satisfactory operation is facilitated.

The author states that 85-lb. rails, in 60-ft. lengths, and of the Am. Soc. C. E. section, were used throughout the construction and reconstruction. The writer is of the opinion that rails in 60-ft. lengths are not altogether desirable on open railway work. It is true that there are fewer joints and more of a continuous track with a longer rail, but there is a disadvantage in handling and difficulty in maintaining proper alignment. Unless special expansion joints are used, 60-ft. rails will kink in summer and have open joints in winter. The 33-ft. rail seems to work to the best advantage for open track, and the 60 and 62-ft. lengths for city or paved track.

From one of the photographs it appears that the rail in paved city streets is also of the T-section. The advisability of using the T-rail in paved streets and adopting it as a standard in all street railway work has been strongly advocated by many railroad men. The Way Committee of the American Street and Interurban Engineering Association will make this subject its principal work during 1909, and it is hoped that a definite recommendation will be offered at its next annual convention.

A number of authorities, including the engineers of cities, are cooperating with the street railways in permitting the installation of T-rails in the streets. Several of the up-State New York cities, including Utica and Syracuse, laid T-rails during 1908, and they are reported as very satisfactory; and such cities as Minneapolis, with T-rails and brick paving, claim to have some of the finest street-railway track and roadway.

Mr. Schreiber.

The great advantage of the T-rail, over any other section, from an operating standpoint, and especially with the increasing wheel loads, is too well known to explain at length, and the question is often only in regard to permission from the authorities for its installation, as they generally object on the ground that the rail does not allow proper paving facilities, and that the paving wears too rapidly along the gauge line of the rail.

It is indeed unfortunate that in the early installation of T-rails in city streets, the paving, especially adjoining the rail, was not properly executed. This placed the T-rail in disrepute for municipal work, so that it is not uncommon for the engineer of a small borough to demand the installation of a Trilby rail, similar to that required in Philadelphia or New York. The writer has often found this to be the case where vehicular traffic was practically nil, and where the paving was macadam. Here the T-rail would not only be cheaper and a great deal more desirable by the railroad company than any other of the common girder sections, but would give better service and more satisfaction to the public at large. This paving about such a rail is as easily maintained as that adjoining the Trilby rail, and with the Trilby section the broken stone is continually getting into the groove and breaking wheel flanges, causing uncomfortable riding, disagreeable noises, and frequent derailments, not to mention the trouble of maintaining the line and gauge.

On account of the great improvement made in T-rail track construction, it is the consensus of opinion of railway engineers that it is only a question of time when municipal authorities will co-operate with railway companies in the adoption of the T-rail, except in the very largest cities, where the streets are narrow, and traffic is very heavy. Just how far the T-rail may be used in densely populated cities, and with heavy wagon traffic and narrow streets, where trucks are compelled to follow the tracks, is difficult to anticipate; but, for ordinary city and interurban railways the T-rail should certainly displace the girder section.

The great difficulty of maintaining track and keeping it in line, and with proper joints, with the Trilby and tram sections, are objections which must be overcome; and there are railway managements which, in all probability, will direct their efforts in the future to satisfying the municipal authorities, by building better track, with substantial foundation, proper drainage, and more improved pavements to suit the T-rail, rather than attempt to operate with an unsatisfactory rail that will meet the present paving conditions.

Mr. Boucher.

WILLIAM J. BOUCHER, ASSOC. M. AM. SOC. C. E.—This paper is interesting in several particulars, and draws one's attention to the very extended development of cross-country electric roads competing with steam roads in Ohio and Indiana, many of which publish time-tables and schedules, run "limited" trains, and traverse distances from 50 to

Mr. Boucher. 100 miles on one division. As these roads are primarily to connect centers of population, they necessarily run through sparsely settled districts, and frequently on private right-of-way; the query then arises, why are so few roads operated by the "third-rail"?

In any consideration of the cost of installation, it is necessary to specify the kind of work wanted. Single-track, span-wire trolley construction may be had at a variety of costs, but, for similar operating conditions and equivalent current capacities, there is a marked difference in the cost of overhead and third-rail construction. Assuming, then, that first-class construction in both cases is called for; also similar operating and current conditions; a double-pole, single overhead trolley with feed wire; other material and labor necessary for erection may be had at a cost of from $4 600 to $5 000 per mile, depending considerably on the nature of the ground in which the poles are placed. An equivalent third-rail system, using an 80-lb. rail on extended ties and insulators, with splice-plates, bonds, and underground cable for road crossings, will cost, erected, about $3 800 per mile of single track. The third-rail is unprotected. Board protection, similar to that installed in the New York Subway, will cost about $2 000 per mile. The maintenance charges should be considerably less for the third-rail system over a period of years.

Noticing, then, this difference of $800 to $1 200 per mile in favor of the third-rail system, there must be some compelling reasons why the overhead trolley is used so much more frequently. Snow should not cause trouble to a third-rail if the interval between cars is no greater than 30 min., and during a storm any road must be prepared to run all night in order to keep the road open. Sleet, however, may cause trouble, but a scraper has been devised and is in use in New York which seems to overcome the difficulty. Of course, in the case of roads operating through country and towns, the frequent shifting of shoe and trolley would prove a nuisance, especially if a double-throw switch must also be operated, and without the latter the shoe is dangerous to passengers who congregate around the steps and near the trucks while waiting to board the cars. In the case of long-distance roads, however, with expanses of country and few towns, and running alongside a platform at stations, the third-rail would seem to be superior. If the road on its private right-of-way is properly fenced, and cattle guards are used at crossings, but little trouble will be caused by cattle and horses.

The following roads are now operating with the third-rail: Albany and Hudson; Wilkes-Barre and Hazleton; Lackawanna and Wyoming Valley; Aurora, Elgin and Chicago; Scioto Valley, Ohio.

MEMOIRS OF DECEASED MEMBERS.

NOTE.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

MARTIN WILLIAM MANSFIELD, M. Am. Soc. C. E.*

DIED SEPTEMBER 25TH, 1908.

Martin William Mansfield was born at Ashland, Ohio, on November 19th, 1850. His father, Martin H. Mansfield, was of English birth; his mother was Anna Saiger, of Mifflin, Pennsylvania.

Mr. Mansfield was graduated from Rensselaer Polytechnic Institute with the Class of 1871. In September of the same year, he entered the service of the Pennsylvania Lines West of Pittsburgh as Assistant Engineer in the Maintenance-of-Way Department on the Cumberland and Muskingum Valley Railroad (a subsidiary line), at Zanesville Ohio. He was promoted successively to Engineer of Maintenance of Way, Superintendent, and Assistant Chief Engineer, which position he held at the time of his death.

On June 24th, 1878, Mr. Mansfield married Miss Carrie Sampsell at Ashland, Ohio, who survives him, with their son, Sampsell W. Mansfield, and daughter, Miss Corinne S. Mansfield.

As a student at Troy Mr. Mansfield was diligent, earnest, and successful. One of his classmates writes of him, "he gave evidence at that time of the unusual talent, that crowned his later years, for working out difficult and abstruse mathematical problems." This talent was indeed characteristic of the man, and was frequently called into play by special lines of investigation assigned to him by his superior officers, who recognized his ability to analyze a mass of apparently heterogeneous facts, reduce them to order, and find the underlying fundamental principle.

Kindly, affable, and accessible, but strict in dicipline, he commanded the esteem and good-will of his subordinates. Earnest, conscientious, and upright, in all things, he had the confidence of his superiors. Quiet and unassuming in manner, cheerful in disposition, and equable in temper, he won the respect of all who came in contact with him.

Mr. Mansfield was elected a Member of the American Society of Civil Engineers on July 5th, 1882, and by his death the Society loses one whose professional abilities and private character were an honor to it.

* Memoir prepared by Thomas H. Johnson, M. Am. Soc. C. E.

MARK WILLIAM SCHOFIELD, M. Am. Soc. C. E.*

DIED NOVEMBER 27TH, 1908.

Mark William Schofield was born in Smithfield, Rhode Island, on November 10th, 1846. His parents were of English stock, and came to the United States about the year 1844. His father died while he was very young, and his mother some years later, in 1864. During his early life he attended the village school in Georgiaville, and later, the Lapham Institute, in Scituate, an advanced academy where many men prominent in later life received a large part of their education.

While attending school his energetic nature led him to spend a portion of his time in the mill in his native town, where he acquired much practical information regarding the details of cotton machinery. His tastes, however, led him toward the profession which he afterward pursued, and early in 1867, and previous to July, 1868, he was for a time with Mr. William S. Haines, and with Cushing and DeWitt, two of the older surveying and engineering firms of Providence. In July, 1868, Mr. Schofield went West, and was engaged on the surveys of the Cairo and Vincennes road, with which General Ambrose E. Burnside was at that time closely identified. Desmond FitzGerald, Past-President, Am. Soc. C. E., was then in charge of the party of which Mr. Schofield was a member.

Late in the fall of 1869 he returned to Rhode Island and re-entered the office of the late Samuel B. Cushing, Sr., M. Am. Soc. C. E., in Providence, where, with the exception of about two years spent on the Northern Pacific Railroad, he remained until the death of Samuel B. Cushing, Jr., M. Am. Soc. C. E., which occurred in 1888. He then carried on the business as the successor of the Cushings, until his death in November, 1908.

In 1869 and 1870 he was leveler on the preliminary survey for the Milford and Lowell Railroad, of which the elder Mr. Cushing was Chief Engineer. In 1873-1874 he was Engineer in charge of the construction of the East Providence branch of the Providence and Worcester Railroad. In the spring of 1881 Mr. Schofield again went West, and, until September, 1882, was Assistant Engineer on the Yellowstone Division of the Northern Pacific Railroad, his section lying between Billings and Miles City, Montana. Here he served with great credit, and, by his faithful and painstaking work, won the confidence and respect of those in the Chief's office. After the completion of the Northern Pacific Railroad, Mr. Schofield resided in Providence and conducted a conservative engineering business of a general nature, doing active work up to within three weeks of the time of his death.

*Memoir prepared by W. H. G. Temple, M. Am. Soc. C. E.

His whole life was marked by that sterling character, unswerving honesty, and strict loyalty to the interests of his clients, which won the respect of all with whom he had either business or social relations, and he unquestionably filled the essential requirements of an honorable man.

Mr. Schofield was married on December 18th, 1873, to Annie S. Brown, a descendant of Chad Brown, one of the early landowners of Providence. His widow, together with four children, survives him.

Mr. Schofield was elected a Member of the American Society of Civil Engineeers on May 1st, 1907.

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AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

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STEEL SHEETING AND SHEET-PILING

By L. R. Gifford, Assoc. M. Am. Soc. C. E.

To be Presented March 17th, 1909.

The subject of steel sheeting and sheet-piling, for some time past, has been receiving a great deal of attention from both engineers and contractors, and the many advantages of these materials, in comparison with the types of timber construction customarily used, is readily appreciated.

The general idea of metal piles is not new, but only within the last five or six years has any considerable progress been made in developing a form which was commercially practical and at the same time sufficiently economical to permit of its use except in special cases.

The earlier forms were of cast-iron, and a very interesting paper, by Mr. Michael A. Borthwick, published in 1836 and reprinted in "Ordinary Foundations," by Charles Evan Fowler, M. Am. Soc. C. E., gives drawings of a number of these forms, and describes their use at considerable length.

That cast-iron sheet-piling did not continue to be used to any great extent was doubtless due to its greater cost as compared with timber sheet-piling, as well as its general unsuitableness for work of this

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

character. It is rather remarkable, however, when it is considered that the difficulty of securing a satisfactory sheet-piling is one of the very troublesome problems with which the construction engineer has to deal, that other forms of metal sheet-piling were not devised, particularly in view of the apparent success which attended the use of the cast-iron piling. When cast-iron piling was first used, however, there was a much greater difference between the cost of metal and timber piling than at present, particularly in the United States, where timber was abundant.

At the present time there are a number of forms of steel piling on the market, and the many advantages which they possess have given them a deserved popularity, which has undoubtedly been augmented by the rapidly increasing cost of timber.

The writer has recently had occasion to study steel "sheeting" and "sheet-piling" quite carefully, and, in presenting briefly the result of these studies to the Society, does so in the hope that the subject may prove of sufficient importance to bring out a further discussion.

The best definitions of the terms "sheeting" and "sheet-piling" are to be found in the paper* by J. C. Meem, M. Am. Soc. C. E., entitled "The Bracing of Trenches and Tunnels, with Practical Formulas for Earth Pressures." Mr. Meem defines "sheeting" as that class of sheathing which is set in or driven coincidentally with the excavation; and "sheet-piling" as that class of sheathing which is driven ahead of the excavation, or beyond its final limits.

In what follows, an attempt will be made to determine the essentials of a satisfactory steel piling, and an analysis of some of the principal sections in use at the present time will be made.

The construction engineer's requirements for sheeting and sheet-piling are about as follows:

First. Strength, to resist earth or water pressure after the piling is in place, without an excessive amount of bracing;

Second. Stiffness, to enable it to be driven without buckling or springing under the blows of the driving hammer;

Third. Water-tightness, to prevent seepage.

On the other hand, the manufacturer will require a section which can be manufactured practically and economically, in order that the first cost shall not be excessive.

Transactions, Am. Soc. C. E., Vol. LX. p. 1.

The various forms of steel sheet-piling which are most used at the present time can be placed in three groups, as shown by the sections on Fig. 1. For convenient comparison, these sections are drawn to the same scale.

The first of the assumed requirements—that the piling must have lateral strength to support earth or water pressure—will now be considered. It will be seen that from this point of view the problem stated generally is to support a continuous and uniformly varying load over a flat surface, and the most convenient measure of the strength is the "section modulus," on the basis of 1 ft. width of the piling in place. This method involves the assumption that the sections when interlocked act as a unit. The diagram, Fig. 2, shows graphically how certain of the sections shown on Fig. 1 compare in this respect.

In constructing the diagram the vertical ordinates are the weight per square foot of the piling assembled, and the horizontal ordinates are the section moduli, expressed in cubic inches for a section 1 ft. in width. An inspection of the diagram will indicate the relative economy of the sections as regards the ratio of weight to strength.

The Lackawanna type would doubtless show approximately the same section modulus as the United States type, as the over-all dimensions of the flanges are about the same. The spring-lock type, as will be noted from an inspection of the drawings, depends on the material in the joints for its lateral strength, but, as the plate forming the body of the pile may be stiffened by standard rolled sections riveted thereto, it is difficult to include this type in the comparison.

The second requirement—that of stiffness to resist buckling while being driven—can be measured by the least radius of gyration of a single piece to be driven, and this is shown graphically on the diagram, Fig. 3. In this diagram the vertical ordinates, as in the previous diagram, represent the weight per square foot, while the horizontal ordinates are the least radii of gyration, in inches.

The third requirement—that of water-tightness—is met most economically by those sections which do not require auxiliary packing pieces, such as wood fillers at the joints, or caulking strips, to render them tight.

From the manufacturers' standpoint, the sections illustrated under Group 1 are desirable for the reason that they are composed of commercial structural shapes, which feature, however, is to a certain extent

GROUP 1.—STANDARD STRUCTURAL SHAPES
13¼″
15″ Channels
4″ z Bars
FRIESTEDT
13½″
15″ Channels
4″ z Bars
FARGO
11″
12″ Beams
12″ Channels
JACKSON
12″
12″ Beams with Clips at intervals
JACKSON
12″
12″ Beams with Special Castings at Top and Bottom
NYE
GROUP 2.—SPECIAL ROLLED SHAPES
12″
UNITED STATES
12¾″
LACKAWANNA
GROUP 3.—SPECIAL SHAPES
12″
WEMLINGER CORRUGATED SINGLE
14″
14″
WEMLINGER CORRUGATED DOUBLE
20″
SPRING LOCK

FIG. 1.

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12″ Beams
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NYE
GROUP 2.—SPECIAL ROLLED SHAPES
12″
UNITED STATES
13¾″
LACKAWANNA
GROUP 3.—SPECIAL SHAPES
12″
WEMLINGER CORRUGATED SINGLE.
14″
14″
WEMLINGER CORRUGATED DOUBLE
20″
SPRING LOCK

FIG. 1.

offset by the fact that considerable shop work is necessary in order to rivet the various pieces together.

Special rolled shapes, Group 2, were designed in order to eliminate all shop work and permit of the sections being used as they come from the rolling mill, particular attention being paid to the interlocking feature.

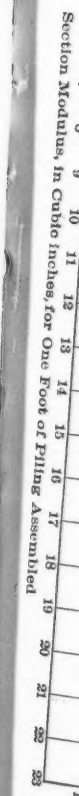
The sections classified under Group 3 require considerable shop work, and are undoubtedly the more expensive of the various types on the basis of cost per pound of material, but this is offset by the fact that a much lighter and stronger section can be secured, which results, generally, in a lower cost per square foot of material in place, as compared with other types.

At the present time no satisfactory method is in general use for specifying the requirements of steel sheeting and sheet-piling. One method is to specify a minimum thickness; another is to specify a minimum weight per square foot after assembling. Neither of these methods is satisfactory, as both entirely overlook what, in the writer's opinion, are the prime essentials, viz., strength, stiffness, and water-tightness.

It is evident that the specification of a minimum thickness can be met by using a flat plate, but this, of course, would be entirely unsuitable, unless some means of properly stiffening the plate is provided, in order to enable it to be driven into place, as well as a proper method of joining the plates at the edges. The requirement of a minimum weight per foot can be met by increasing the weight at the joint and reducing the thickness of the metal between the joints, which is equally undesirable.

The writer considers that a proper and reasonable specification for steel sheeting and sheet-piling is essential and should be used by engineers whenever this material is to be used.

The problem of computing the strength required in sheeting and sheet-piling is a very difficult and complicated one, as is well illustrated by Mr. Meem's paper, previously referred to, and the discussion in connection therewith. It is very necessary, however, that any specifications should take this very important feature into consideration, and the following method, it is believed, will provide a satisfactory basis for specifying the minimum strength required.



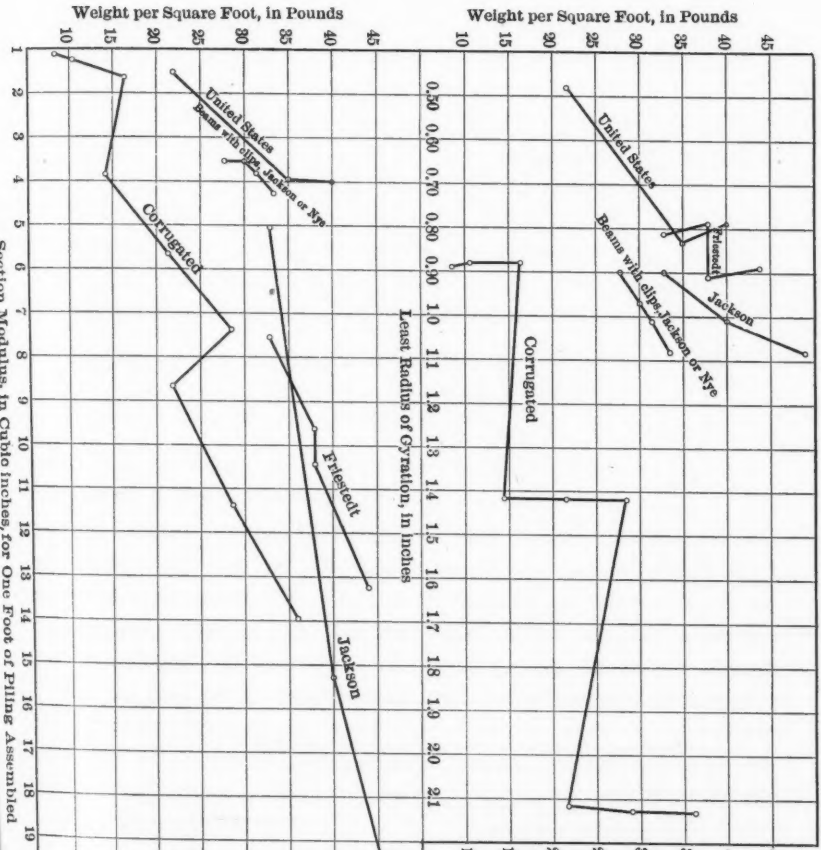

Weight per Square Foot, in Pounds
United States
Friestedt
Jackson
Beams with clips, Jackson or Nye
Corrugated
Least Radius of Gyration, in inches

FIG. 3

Weight per Square Foot, in Pounds
United States
Beams with clips, Jackson or Nye
Corrugated
Friestedt
Jackson
Section Modulus, in Cubic inches, for One Foot of Piling Assembled

FIG. 2

The assumptions made are:

First. The pressure to be supported is hydrostatic;

Second. The sheeting or sheet-piling is in the condition of a simply supported beam, one support being the bracing at the top, and the other the ground at the bottom;

Third. The load is uniformly increasing from zero at the top to a maximum at the bottom.

Fourth. For permanent work, the allowable fiber stress should not exceed 16 000 lb. per sq. in., which provides a safety factor of 4; for temporary work, however, a fiber stress of 20 000 would not be excessive, and this provides for a safety factor of approximately 3.

Expressing the foregoing assumptions in a formula, we have:

$$S = \frac{4.01\ d^3}{16\,000} \text{ for permanent work;*}$$

$$S' = \frac{4.01\ d^3}{20\,000} \text{ for temporary construction;*}$$

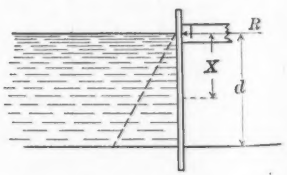
when S and S' = section moduli in inches^3;

d = depth of water supported, in feet.

Proper bracing, of course, is essential, and the values obtained from these formulas are intended merely to provide for a maximum theoretical condition, to be used, as previously stated, as a basis for specifying the minimum strength required. For any special case, exact figures should be made, taking into account the spacing of the rangers and bracing to be used, the character of the soil, etc.

---

*X = distance from support to point of maximum bending,
d = span of beam, *i.e.*, length of piling between supports,
R = reaction,
M = bending moment at distance X,
S = section modulus of beam,
f = allowable fiber stress,
W = total load.



$$W = \frac{62.5\ d^2}{2}$$

$$R = \frac{62.5\ d^2}{6}$$

$$X = \frac{d\sqrt{3}}{3}$$ See "Treatise on the Resistance of Materials" Wood, p. 195.

$$M = \frac{62.5\ d^2}{6} \times \frac{d\sqrt{3}}{3} - \frac{62.5\ d^2}{2} \times \left(\frac{d\sqrt{3}}{3}\right) \times \frac{1}{3d^2}$$

$$= 4.01\ d^3$$

$$M = f\,S = 4.01\ d^3$$

$$S = \frac{4.01\ d^3}{f}.$$

With this explanation of the paragraph relating to the strength, the writer offers the following specifications, which, it is believed, will provide steel sheeting or sheet-piling, without designating any particular form, which will meet any reasonable requirements.

Specifications.

Material.—All steel used shall be in accordance with the Specifications of the American Association of Steel Manufacturers for Structural Steel for Buildings, Revised to February 6th, 1903.

Unit Strains.—The allowable unit strains per square inch in bending on extreme fiber shall not exceed the following values:

For temporary work, 20 000 lb.

For permanent work, 16 000 lb.

Strength.—The section used shall have a section modulus per foot, when assembled in place, of not less than the following:

	Permanent.	Temporary.
Lengths of 10 ft. or less	0.25	0.20
" from 10 to 15 ft.	0.85	0.68
" " 15 " 20 "	2.00	1.61
" " 20 " 25 "	3.92	3.13
" " 25 " 30 "	6.77	5.42
" " 30 " 35 "	10.75	8.60

Stiffness.—Single pieces driven separately shall have a ratio of length in inches to least radius of gyration of not more than 250.

Water-Tightness.—The sections when assembled in place shall be water-tight at the joints, without the use of auxiliary packing or caulking pieces, unless such packing or caulking pieces are made part of and are assembled with the sections of the sheeting or sheet-piling before driving.

Driving.—Care shall be taken in driving to prevent battering of the heads of the piles, and, if necessary to prevent this, a tight fitting cap shall be used. Driving shall be done, preferably, by steam or compressed-air hammers.

Applications.

Steel sheet-piling of rolled sections was first used in the United States in 1902, when the Jackson type of beams and channels was utilized in the construction of a coffer-dam for the foundation of the Randolph Street Bridge, Chicago, Ill. Since that time the use of steel sheet-piling for coffer-dams of all kinds has become quite general.

The largest example is, without doubt, the coffer-dam now under construction for the United States Government at Buffalo, N. Y. This coffer-dam encloses an area 887 by 200 ft., in which is to be constructed a ship lock in Black Rock Harbor and Channel. Two lines of steel sheet-piling of the Lackawanna type are being used, driven 30 ft. apart and connected by cross-walls at intervals of 30 ft. The pockets thus formed are to be filled with clay, before the water is pumped out of the coffer-dam.

A very economical application of steel sheet-piling is its use in connection with dams or embankments, where it is frequently driven to form a core-wall designed to prevent seepage. The first suggestion of its use for this purpose which the writer has seen was that made by the late George S. Morison, Past-President, Am. Soc. C. E., in his paper entitled "The Panama Canal."* In this paper Mr. Morison proposed to use steel sheet-piling of rolled sections in connection with the "Bohio Dam."

Probably the first actual application of steel sheet-piling for a core-wall was made by the Hackensack Water Company, near Hillsdale, N. J., the Friestedt type being used. A recent example is a dam at Scotland, Conn., for the Uncas Power Company, of Norwich, Conn. In this case an attempt was made to drive 6 by 8-in. yellow-pine sheet-piling of the ordinary tongued and grooved type fitted with malleable cast-iron chisel points. The soil was of a dense gravelly character, or, more properly, a hardpan, which the timber piling failed to penetrate, the piles brooming or splitting to such an extent as to be utterly worthless. Steel sheet-piling of the Wemlinger corrugated type, formed of $\frac{3}{16}$-in. and $\frac{1}{4}$-in. plates, was substituted and satisfactorily driven, although in some cases more than 4 000 blows of a steam hammer with a ram weighing 3 000 lb. were necessary in order to secure a penetration of 18 ft., due to the very compact character of the soil.

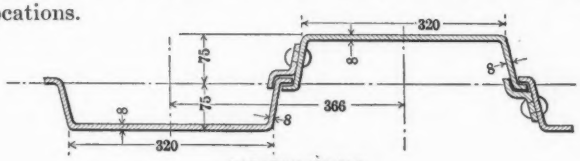
Another application of steel sheet-piling which is becoming more and more general is its use as shoring in the construction of building foundations, especially where the foundations must be carried below those of adjoining buildings or through soft or water-bearing soil. It is frequently possible by the use of steel sheet-piling to secure satisfactory foundations which would otherwise require the use of pneumatic caissons, which are very expensive.

The construction of mine shafts is another application in which

* *Transactions*, Am. Soc. C. E., Vol. L, p. 177.

steel sheet-piling has been used successfully. However, probably the largest field for the adoption of steel sheet-piling is in trench work, and, while it has been used to some extent for this purpose, it is only recently that a type of steel sheeting has been developed which is economical in comparison with wood sheeting. This is due to the fact that it is now possible to secure a steel sheeting weighing 10 lb. or even less per sq. ft. which can be driven in widths of 12 in., and when it is considered that this sheeting, with proper care, may be used practically indefinitely, the saving over the timber is evident.

Recently, in the construction of a sewer at Summit, N. J., 100 lin. ft. of 10-ft. Wemlinger corrugated sheeting was used on 8 000 ft. of sewer in a trench varying in depth from 9 to 22 ft., that is to say, the same sheeting was driven and pulled at least 80 times, and at the completion of the job was still in good condition. This sheeting was made of 16-gauge material, weighing 5 lb. per sq. ft. in place. Sheeting of the same type is now being used for sewer and conduit trenches in Washington, D. C., Buffalo and Rochester, N. Y., and a number of other locations.



LARSSEN PILING
Dimensions in Millimeters

FIG. 4.

A use to which the lighter forms of steel sheet-piling are especially adapted is in connection with levees. Driven down through the center of these it would effectually cut off any seepage under them and absolutely prevent their being undermined by crawfish, muskrats, or other boring animals. The cost of the steel sheet-piling used in this manner would be largely made up in the saving which could be effected by the elimination of the muck ditches and the greatly reduced section of the levee itself made possible thereby.

Still another application is in the construction of docks and bulkheads, and, while it has never been used for this purpose in the United States, as far as the writer is aware, it has been done in Germany, at Hohentor Harbor, near Bremen, and in the port of Bremen, where a considerable quantity of bulkhead work has been built of the Larssen type of rolled-steel piling, Fig. 4.

The Larssen type of steel sheet-piling was also used in the construction of the walls of two large ship-locks on the canal above Bremen, near Hamelingen.

In all these cases, the steel sheet-piling was driven along the bank, and longitudinal waling-beams were afterward bolted to the piling, and the entire structure was secured near the top with anchor-rods fastened at one end through the waling beams and at the other to concrete slabs buried in the bank. The results are entirely satisfactory; the bulkheads present a very substantial appearance, and are much more economical in cost than concrete or masonry structures, while, at the same time, they are considerably more durable than timber bulkheads.

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PAPERS AND DISCUSSIONS

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THE SIXTH STREET VIADUCT, KANSAS CITY.

By E. E. Howard, Assoc. M. Am. Soc. C. E.

To be Presented April 7th, 1909.

The Sixth Street Viaduct, recently built, is a steel structure, about 8 000 ft. long, providing an elevated roadway and two car tracks across the broad, flat valley of the Kaw River to the high land on each side. It was designed to accommodate, and, by improved conditions for comfort, safety, and speed, to facilitate street-car and vehicular traffic; thus contributing to the unification of the social and commercial interests of Kansas City, Missouri, and Kansas City, Kansas.

The main viaduct and those portions of the side laterals or approaches now finished include, in general figures, 1 900 lin. ft. of roadway 38 ft. wide, 6 100 lin. ft. 30 ft. wide, 1 000 lin. ft. 24 ft. wide, and 1 000 lin. ft. 20 ft. wide, a total of 10 000 lin. ft. of roadway; also 8 500 lin. ft. of double-track railway. In this construction there have been used more than 120 000 lin. ft. of piles, 23 000 cu. yd. of mass concrete, 13 000 tons of steel, nearly 10 000 cu. yd. of reinforced concrete, 34 000 sq. yd. of asphalt pavement, besides minor and miscellaneous material. On one lateral, or side approach, where an incline could not be conveniently placed, there were installed two large electric traffic elevators of sufficient capacity to raise any vehicle and team.

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

Figs. 1 and 2, Plate II, are general views of the viaduct and the bridge.

Kansas City is located on the south side of the Missouri River, below one of its great bends, and lies in the States of Missouri and Kansas, with the crooked Kaw River flowing through it to join the greater stream. While commercially and socially possessed of common interests, the community has political and governmental divisions, due to its position on the border line of two States, and is divided into two distinct cities with their various suburban settlements.

The valley of the Kaw River, popularly called the "West Bottoms," is about a mile wide and is quite flat; it is skirted on each side by high bluffs and hills, while the channel of the river meanders through it, in its lower reaches lying close to the western bluff. The Missouri River has been gradually pushed northward, through the agency of dikes and mattresses, and has deposited, in the last fifteen years, a strip of land, about one-quarter of a mile wide, extending from side to side of the valley of its tributary. While practically the entire residence portions and the principal retail business districts of both cities are on the hills, there are to be found, crowded into the Kaw Valley, many commercial industries, packing houses, stock yards, manufacturing plants, wholesale warehouses, the freight stations of nearly all the thirty-four railroads entering the city, several large terminal freight yards, and the present union passenger station. As an essential adjunct, there is, throughout the bottoms, a complicated network of railway tracks, occupying streets and alleys as well as private rights of way.

The transaction of the business of the cities involves an enormous traffic into this valley from both sides, and all traffic from city to city must pass through it. A very large portion of the freight consigned for local consumption must be hauled out of the bottoms; some up-town distributing houses haul goods out, store them, and then haul them back; the packing houses, factories, and other industries are continually conveying their products to both cities; and, too, a great deal of the merchandise consumed in Kansas City, Kans., is hauled from Kansas City, Mo.

All this traffic is confronted by very inadequate passageways and very heavy grades from valley to hills, not less than 5% on the east side, and not less than 6% on the west side. Furthermore, on the east side there are from the northern part of the city only two possible

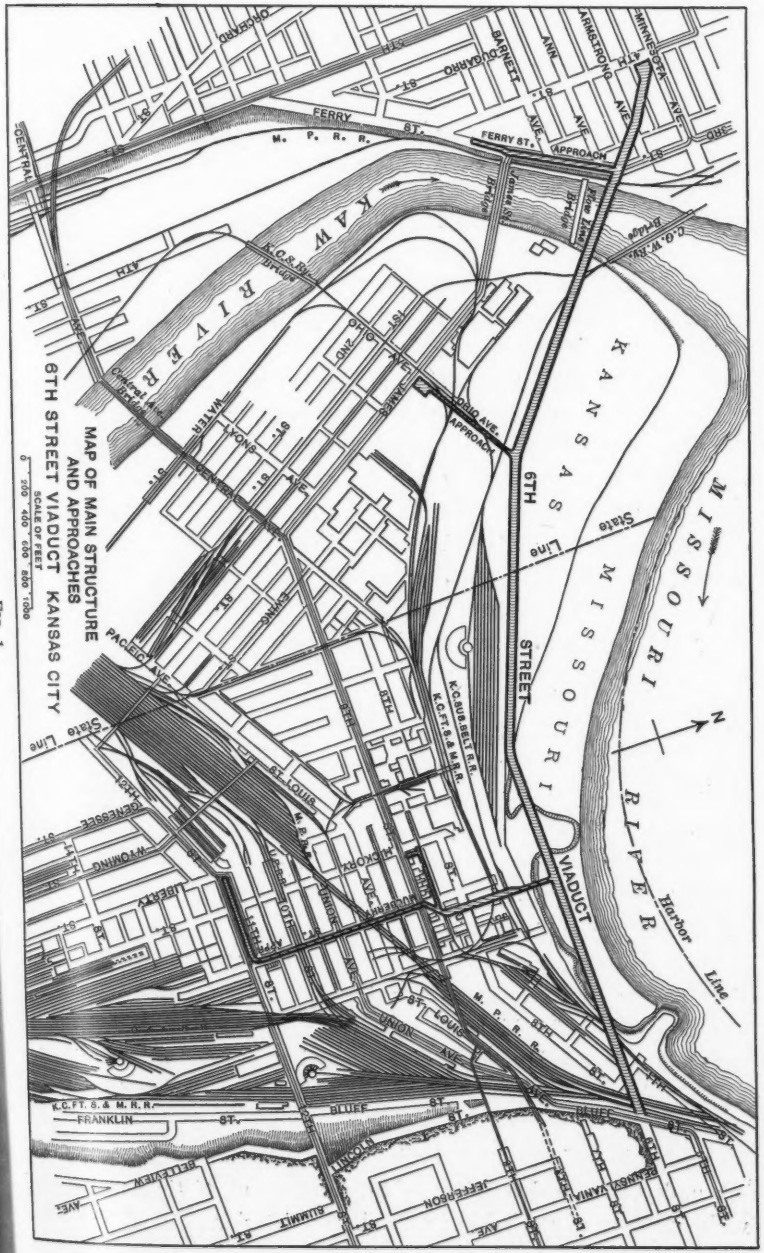

MAP OF MAIN STRUCTURE AND APPROACHES
6TH STREET VIADUCT KANSAS CITY
SCALE OF FEET
FERRY ST.
M. P. R. R.
KAW RIVER
KANSAS
MISSOURI
RIVER
6TH STREET VIADUCT
6TH Line
State Line
Harbor Line
PACIFIC AVE.
K.C.FT.S.& M.R.R.
K.C.SUBURBAN R.R.
ST. LOUIS AVE.
UNION AVE.
BLUFF ST.
FRANKLIN ST.
OHIO AVE. APPROACH
ARMSTRONG AVE.
ANN AVE.
BARNETT AVE.
OHIO ST.
CENTRAL
WATER
WYOMING
GENESSEE
LIBERTY
JEFFERSON
JEFFERSON AVE.
VANNIA
BENNETT
LINCOLN
BELLEVIEW
SUMMIT
N

FIG. 1.

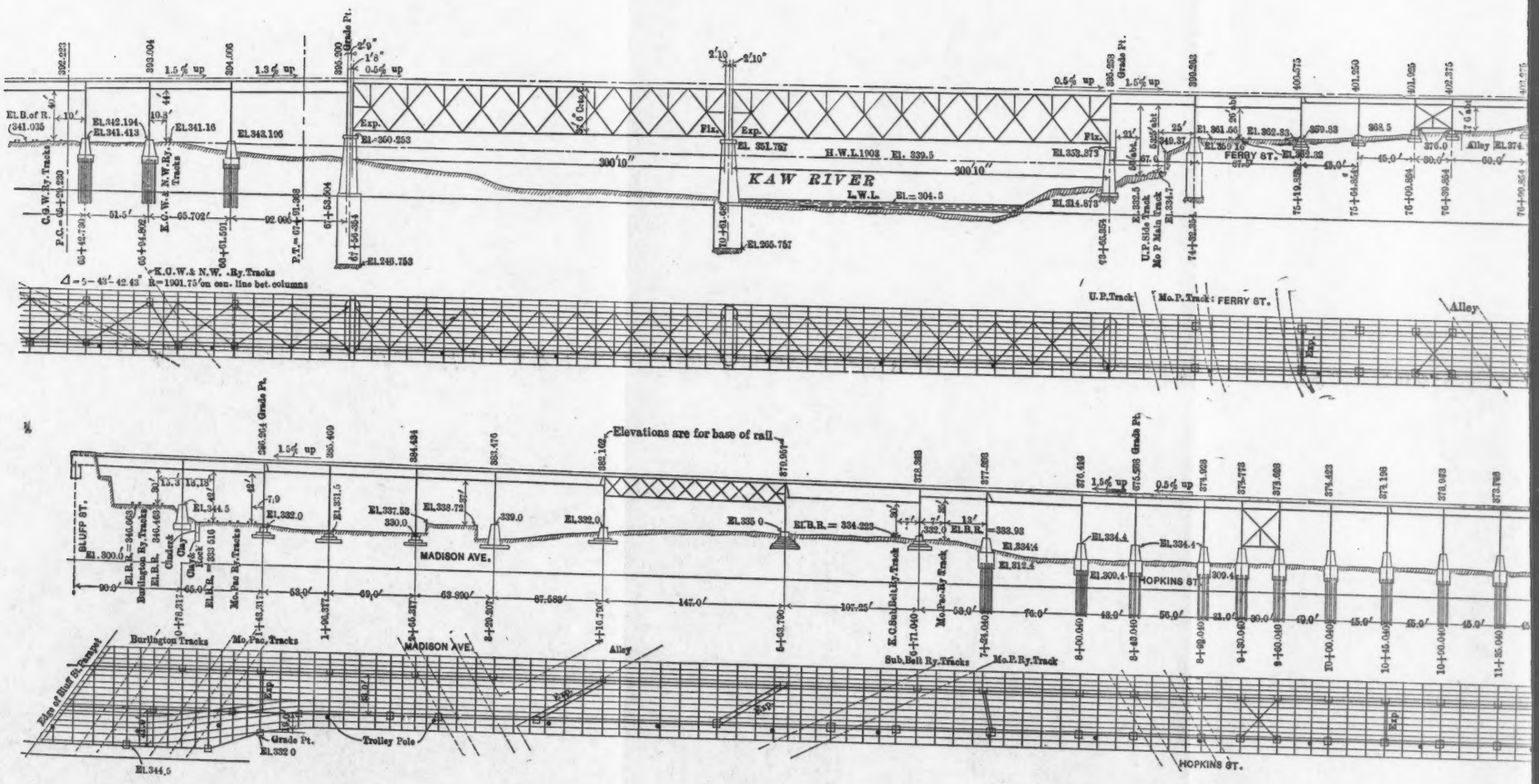
entrances to the bottoms; and, on the west side, giving access to Minnesota Ave., the principal business street of Kansas City, Kans., there is only one passageway. In addition to the steep grades to be contended with, the vehicular route to every freight station is over dangerous grade crossings frequently congested with traffic, while the interurban travel is compelled to follow circuitous routes.

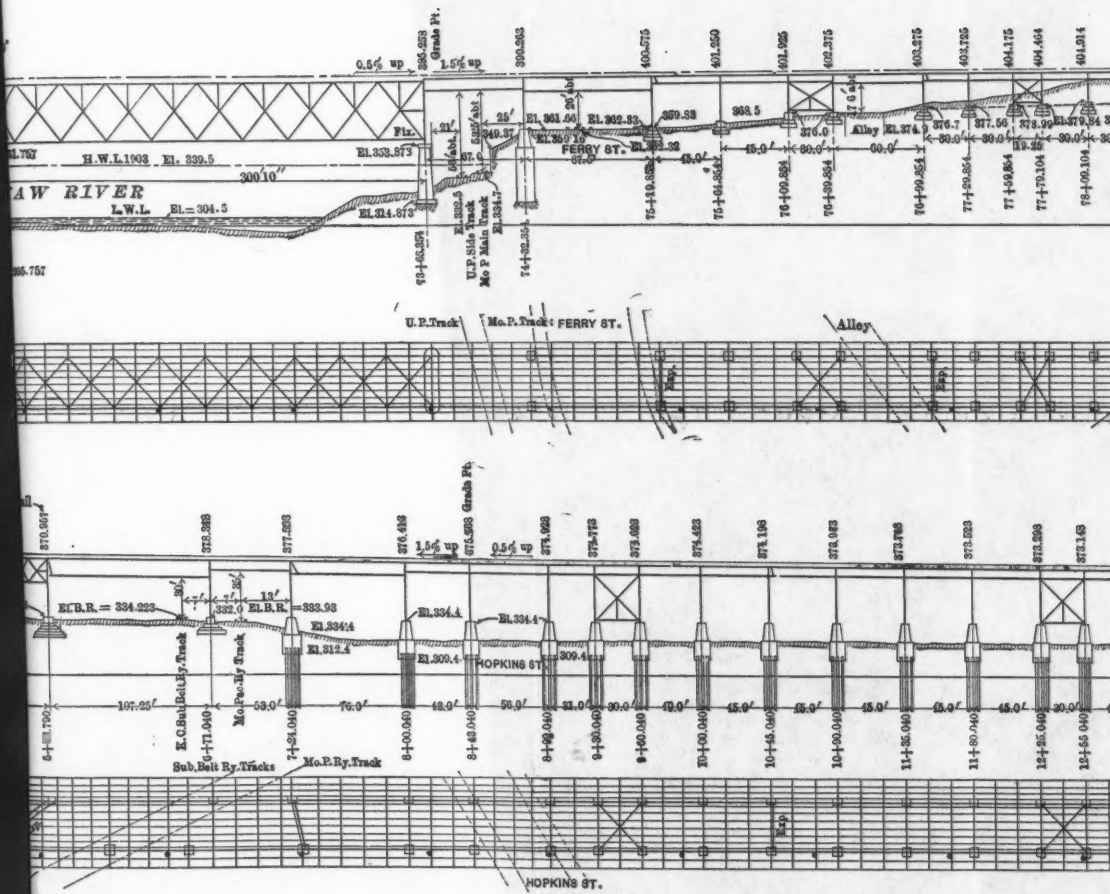
Not only does the highway traffic to and from the bottoms operate under adverse conditions, but the street railway facilities are inadequate, there being only two through lines to handle the entire between-city travel. One of these is a very circuitous surface line, operated at great discomfort because of the many grade crossings, the steep grades, and the congestion of streets in the bottoms. The other line is elevated for a considerable part of the way, but the route is indirect. The need of additional rapid transit across the bottoms was long keenly felt by the street railway interests and the public, especially the latter.

Various measures have been devised from time to time to expedite this travel, and a viaduct has often been advocated. Shortly after the disastrous flood of 1903, which, by the inundation of the entire west bottoms, for a time completely cut off all intercommunication, a plan was proposed for the two cities to build jointly a street-car and vehicular viaduct, but it soon became apparent that no money was or would be available for such a purpose, and in any case the difficulties of two city governments jointly handling a structure located in two cities, two counties, and two States, seemed almost insuperable.

The proposition to build a toll viaduct by private capital for an investment was then broached, and investigation was undertaken to determine the feasibility of such a plan. A preliminary survey showed that a viaduct could be built across the accreted lands near the Missouri River, then unoccupied, which would shorten the distance from center to center of business districts by street car 4 150 ft. (26%) and 18 min. (60%); would give easy grades and uninterrupted passageway from city to city for highway travel; and, by a judicious location of laterals and approaches, could give convenient access, with elimination of all grade crossings, to each of the various freight stations in the bottoms.

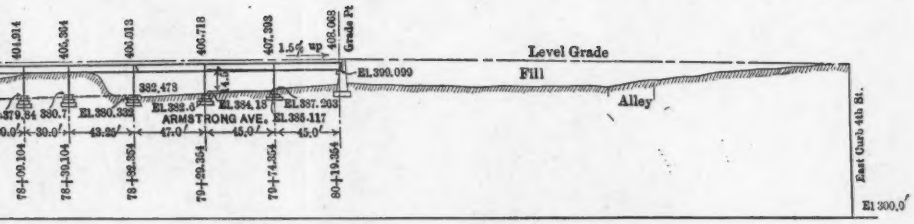
Careful investigation, extending over a number of months, was then undertaken to determine: First, the amount of time to be saved by the patrons of a viaduct; second, the deterioration of equipment


KAW RIVER
L.W.L. El. = 304.5
H.W.L. 1908 El. 339.5
El. 351.757
El. 350.253
El. 365.757
El. 245.753
El. 314.873
El. 333.873
El. 332.5
El. 335.7
El. 361.60
El. 359.16
El. 362.53
El. 362.32
El. 369.33
El. 368.5
El. 376.0
El. 374.
El. B. of R. (341.035
El. 342.194
El. 341.413
El. 341.16
El. 343.106
El. 344.5
El. 332.0
El. 331.5
El. 337.53
El. 338.72
El. 330.0
El. 339.0
El. 332.0
El. 335.0
El. B. R. = 334.223
El. B. R. = 333.93
El. 334.4
El. 334.1
El. 312.2
El. 309.4
El. 309.4
El. 300.0
El. B. R. = 345.063
El. B. R. 346.463
El. B. R. = 333.516
Grade Pt.
Exp.
Fix.
Elevations are for base of rail
Δ = 5° 43' 42.43"
R = 1001.75' on cen. line bet. columns
K.C.W.& N.W. Ry. Tracks
C.B.W.Ry. Tracks
K.C.W.& N.W.Ry. Tracks
U.P.Track
Mo.P.Track
U.P.Side Track
Mo.P.Main Track
FERRY ST.
Alley
BLUFF ST.
Burlington Ry. Tracks
Clinton
Clay
Clay Lick
Mo.Pac.Ry.Tracks
MADISON AVE.
HOPKINS ST.
Sub.Belt Ry.Tracks
Mo.P.Ry.Track
Burlington Tracks
Mo.Pac.Tracks
Face of Bluff St. Retaining Wall
Grade Pt. El. 332.0
Trolley Pole
El. 344.5
P.C. = 60+29.230
P.T. = 67+51.268
P.C. = 60+42.730
60+94.892
60+61.591
65+53.004
67+56.354
65.702'
92.066'
51.5'
300'10"
2'10"
2'9"
1'6"
0.5% up
1.5% up
1.3% up

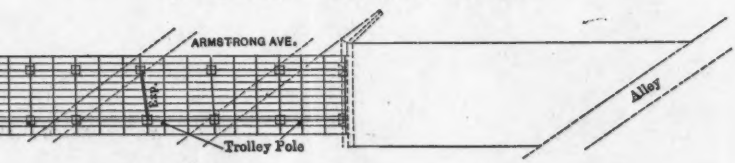

0.5% up
Grade Pt.
1.5% up
H.W.L.1903 El. 339.5
L.W.L. El.= 304.5
El. 353.873
El. 314.873
300'10"
AW RIVER
Fix.
25'
El. 361.56
El. 362.33
359.33
368.5
FERRY ST.
El. 362.32
Alley El. 374.9
U.P. Side Track
Mo. P. Main Track
El. 334.1
U.P.Track
Mo.P.Track
FERRY ST.
Alley
Exp.

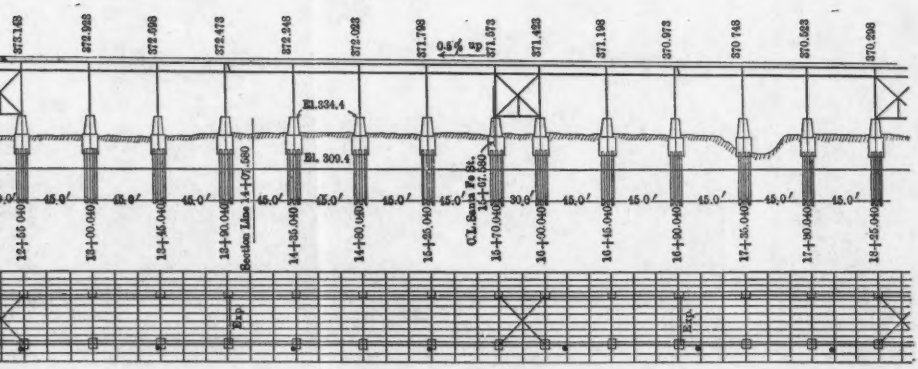
Grade Pt.
1.5% up
0.5% up
El.B.R. = 334.223
El.B.R. =333.93
El.334.4
El.312.4
El.309.4
HOPKINS ST.
107.25'
59.0'
76.0'
48.0'
56.0'
31.0'
30.0'
42.0'
45.0'
45.0'
45.0'
45.0'
30.0'
K.C.Bel.Bel.Ry.Track
Mo.Pac.Ry Track
Sub.Belt Ry.Tracks
Mo.P.Ry.Track
Exp.
HOPKINS ST.

PLATE I.
PAPERS, AM. SOC. C. E.
FEBRUARY, 1909.
HOWARD ON
THE SIXTH STREET VIADUCT, KANSAS CITY.



PORTIONS OF PROFILE AND PLAN
6TH STREET VIADUCT, KANSAS CITY





to be lessened; third, the additional loads to be carried per vehicle; fourth, the actual money to be saved by the user; fifth, the amount of toll the user could afford to pay and still profit by the use of a viaduct; sixth, the probable volume of traffic to patronize such a structure; seventh, the effect of the viaduct on the distribution of population, and the consequent increase of travel; eighth, the probable growth of the cities and the increase of business; ninth, a minute and thorough estimate of the cost of construction; tenth, the annual cost of operation; eleventh, the annual income; and twelfth, the earnings to be realized.

For many of these questions there was no precedent, and some conclusions were based on unsatisfactory data. The determination of the number of vehicles passing to and from the bottoms required merely an extended series of actual counts, secured from day to day, principally by detectives of the Pinkerton service. These counts were made at various times from October, 1903, to June, 1904; and numerous independent tallies agreed closely with a total of about 12 000 vehicles per day to and from the bottoms.

As a further verification of these figures, statements were obtained from leading transfer men and large haulers as to the estimated use they would make of the viaduct, the total sum of which showed an entirely satisfactory figure.

A study of comparative statistics showed Kansas City to have exceptional business prospects, for, while at present twenty-second in population, it ranks as ninth in bank clearings, twelfth in total postal business, second in railroads, second in grain business, and it is situated in the center of a rich and productive territory.

Neither did the estimates of cost of construction and maintenance offer particular difficulty; but the determinations of the individual profit to be secured and the toll which could be reasonably expected were but approximations.

The matter of the utilization of the viaduct for street cars was taken up early with the company operating the cars in the city, and with certain suburban lines, with the result that tentative agreements, later duly confirmed by contracts, were made for traffic arrangements.

The conclusion of the preliminary investigation was that the structure as then planned, reaching from city to city, with laterals at properly selected points giving uninterrupted access to the freight houses

was feasible as an investment, could be built for about $2 275 000, could be operated for about $180 000 per annum, and could earn from 5% upward on the capital stock, depending on the future growth of the city.

The Kansas City Viaduct and Terminal Railway Company was organized to build and operate such a viaduct, and the Common Councils of the two cities granted to the Company the necessary franchises. The principal provisions of these franchises are:

The franchise is to continue in force for thirty years;

A schedule of maximum tolls is fixed;

A clause provides that the City may at the end of ten years purchase the viaduct at any time, by notifying the Company one year in advance, at the actual cost and a stipulated varying interest rate;

A provision is made for the Company to pay the City 2% of its gross revenue.

As soon as certain necessary legislation was obtained from the Missouri Legislature, financial arrangements were completed, right of way was secured, and the construction, for which general contracts had already been made, was begun.

It had been the intention to commence the Mulberry Street Approach first, to finish that approach and the east end of the main structure from the approach to the eastern terminus, and have this portion in operation before the completion of the main viaduct; but the Company's attorneys discovered many difficulties in getting right of way for the approach, so the construction of the Main Viaduct was proceeded with until the legal complications could be untangled.

The following pages contain a discussion of the design and construction in the following order:

1.—Superstructure, design, manufacture, and inspection.
2.—Substructure, design, and construction.
3.—Engineering field methods.
4.—Erection of superstructure metal.
5.—Floors, concrete floor, paving, street-car deck, hand-rail, and lighting system.
6.—Approaches and laterals.
7.—Final costs.

1.—Superstructure.

The design provides a roadway 30 ft. wide over a part, and 38 ft. wide over the remainder of the structure, to carry a loading varying according to span from 50 to 100 lb. per sq. ft., or a 15-ton road roller; and, separated from the roadway by a substantial hand-rail, a double track for electric cars, to support a continuous line of cars on each track, each car weighing 40 tons and having a length of 43 ft. 3 in. over all. Each of these assumed loads is increased by an impact percentage as noted hereafter.

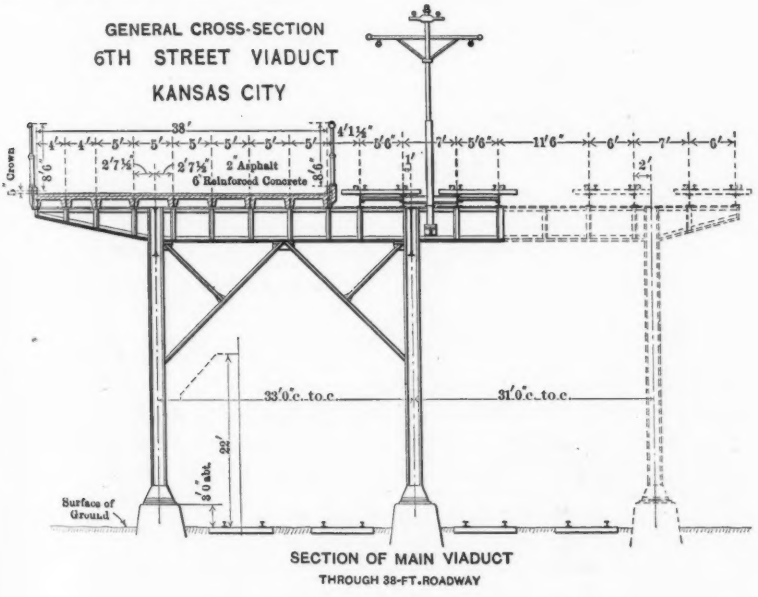


Fig. 2.

Before adopting the layout for the spans, an extended economic study was made by preparing estimates for different arrangements of girders and columns. The varying amount of shopwork, and the unit cost of material, as well as the total weight of metal per linear foot, were duly considered, the resultant variation of substructure cost with span length was estimated, and the most economical girder length, floor arrangement, and tower spacing for the confronting conditions were definitely determined.

This led to the selection of a two-column structure with one column

about under the center of the roadway, and the other about under the center of the motorway, with longitudinal girders riveted into the columns, cross-beams riveted into the girders and columns, and stringers laid on top of the cross-beams.

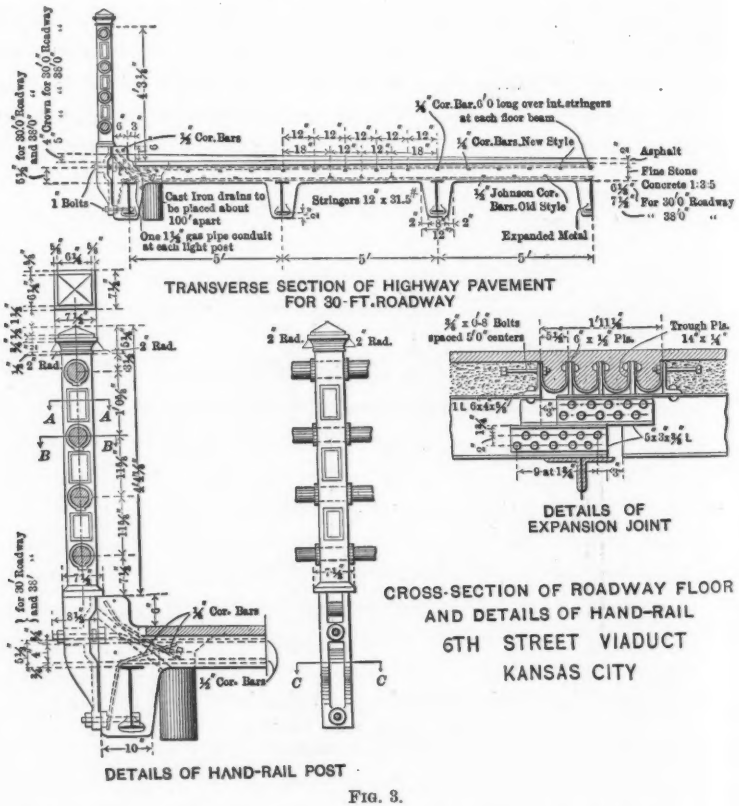


FIG. 3.

Considerations of possible increase of railway traffic resulted in the decision to arrange the viaduct so that at any time there could be added two additional tracks, each to carry a continuous line of cars, each car weighing 70 tons and having a length of 43 ft. over all. This is to be done by adding a third row of columns with girders, and splicing the cross-beams to the original motorway cantilever beams.

The two columns are 33 ft. from center to center transversely, and spaced so as to be loaded equally when the final loading is applied.

The third column will be 31 ft. from the original motorway column and at mid-point between the two added tracks. The transverse bracing is arranged so that tracks may be laid on the ground between the columns, and occasional turnouts are allowed for by spans of special length.

Longitudinally, the structure is divided into sections composed of one span of 30 ft., and seven spans of 45 ft. The 30-ft. spans have longitudinal bracing to care for the entire thrust of the 345-ft. section, thus imposing very few obstructions to free transverse passageway below, a stipulation of certain river-front property holders. The mid-girder between the towers is supported at one end in a pocket or shelf so as to slide and allow for expansion.

The highway floor is of concrete, enveloping the steel stringers, and supporting a pavement of asphalt. Hand-rails are placed on each side of the highway. The car tracks are of ordinary rails with the usual creosoted pine ties and guard-rails. The maximum grade on the main viaduct is 1.5%, and the minimum is 0.5%, this latter being maintained so as to drain the asphalt pavement properly.

The following working intensities were used in the design:

Tension Stresses.

	Pounds per square inch.
Flanges of plate girders (counting in one-eighth of the web)	14 000
Rolled shapes	16 000

Bending Stresses.

Extreme fiber of rolled sections	16 000
Extreme fiber of timber beams	2 000

Compression Stresses.

Top chords	$18\,000 - 70\,\frac{l}{r}$
Inclined end posts	$18\,000 - 80\,\frac{l}{r}$
Intermediate posts and diagonals	$16\,000 - 80\,\frac{l}{r}$
Lateral struts and bracing	$16\,000 - 80\,\frac{l}{r}$
Columns	$16\,000 - 60\,\frac{l}{r}$

Shearing Stresses.	Pounds per square inch.
Webs of plate girders	10 000
Rivets	10 000
Impact, for motorway loading	$I = \frac{400}{L+500}$
Impact, for highway loading	$I = \frac{400}{L+150}$

Under extreme temperatures and full live load, the shortest columns were computed to have an extreme fiber stress of 24 000 lb. per sq. in., but this was not considered excessive for such unusual conditions.

All calculations were based on the use of equivalent uniform live loads instead of actual wheel concentrations. Because of the very considerable labor involved in computing anew, for each slight change in panel length, the cross-beam reactions, the results from a series of panels of different lengths were plotted and a curve was drawn connecting all points. This diagram proved of great service in designing the many special girders, for a principal part of the calculations was saved by taking off directly the amount of the imposed loads.

Trestle Construction.

The various special girders and columns were detailed in conformity with the standard portions of the structure, and may be generally included in the following description.

Stringers.—Both the highway and the motorway stringers consist of I-beams set on top of the cross-girders and cantilevers, and connected thereto by rivets or bolts. Small shims are used under the highway stringers to provide a crown for the roadway. All stringers are continuous for the corresponding girder length, either 30 ft. or 45 ft., and are joined by small splice-plates. At expansion joints, stringers are halved and reinforced with bearing angles so that they slide upon one another. Connecting the motorway stringers, there is a rigid system of bracing, composed of single-angle diagonals, and of cross-struts of either one angle or one channel. The connections for all this bracing are made by lug-angles to the webs of the beams so as to leave their upper flanges clear for convenience in laying the ties.

Cross-Girders and Cantilevers.—Cross-girders are spaced at 15-ft. centers, thus placing two intermediate beams in the 45-ft. girder spans and one in the 30-ft. spans. They are riveted to the webs of the main girders or to the columns, with their upper flanges level with the tops

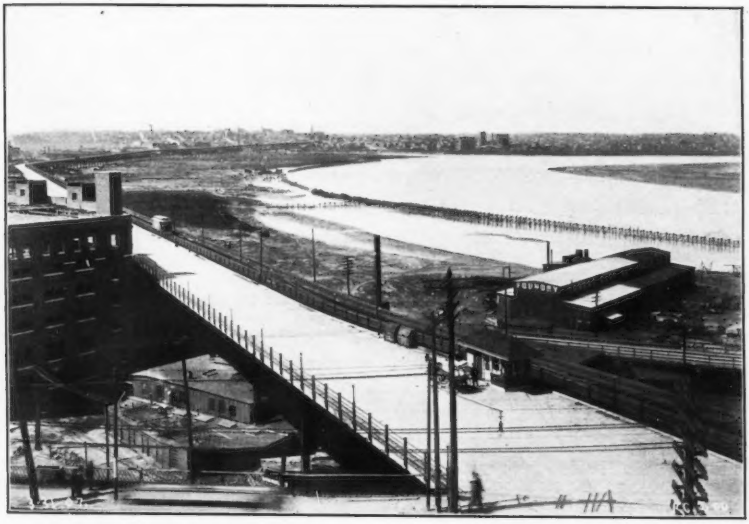


FIG. 1.—THE COMPLETED VIADUCT LOOKING FROM THE EASTERN END TOWARD KANSAS.

FIG. 2.—THE COMPLETED BRIDGE OVER THE KAW RIVER.

of the main girders. These beams are 52 in. deep, with ⅜-in. webs, and flanges composed of two angles. The intermediate stiffeners, which are pairs of angles placed at each concentration point, are crimped, while the end stiffeners have fillers on the web. The cantilevers at each end of each cross-beam follow the same detail, that on the highway side varying in depth from 52 to 12 in., though the motorway cantilever is of full depth to its end, where provision is made for splicing on a future cross-beam. Splice-plates extend over the girders or columns, and connect the top flanges of the cross-beams and cantilevers. At columns, the thrust at the lower flange is carried into the cross-beams with cast-iron thrust-blocks bearing against the channels of the posts and the webs of the main girders. These blocks were cast to a driving fit, were driven to place after the field riveting was completed, and were also supported by bolts through the column webs.

Main Girders.—The standard 45-ft. girders are 72 in. deep, with ⅜-in. webs and duplicate flanges of two angles and cover-plates. The general details are the same as for the cross-beams, except that the intermediate stiffeners are spaced 5 ft. apart.

Columns.—The columns, throughout, are of two rolled or built channels joined by a built I, with the webs of the channels in general parallel to the longitudinal axis of the structure. Stay-plates connect the flanges of the channels. Columns extend clear to the top of the main girders and cross-beams, providing for them full riveted connections. Lug-angles and brackets are riveted to the columns for the connection of the bracing, and shelf-angles are provided for convenience in girder erection. The shoe is built of side-plates and angles, fastened to the channels, and distributing the load to two heavy base-plates. Curved plates, 18 in. high, are fastened to each side, as sleeves for the anchor-bolts, and as bearings for the anchor-nuts.

At expansion points the columns have on one side pockets made of plates riveted to the channels supporting a bottom or base on which the girders slide. The tops of the columns are tied back to the adjacent span in a manner to relieve the main connection rivets of tension due to eccentric loading.

Bracing.—Lateral bracing is used only on curves, and in each tower span, it being considered that elsewhere the concrete floor would give all needful lateral rigidity. Where used, it consists of two diagonals of two angles each, with sufficient area to take the stress in either tension or compression. These angles are placed just below

the cross-beams, are riveted to the column beams and to the main girders, and also to each other beam they touch.

Longitudinal bracing at the towers is made up of two diagonals and a lower horizontal strut, all composed of four angles, laced, I-shaped, and of dimensions to fit into the columns. At the upper points the diagonals are attached also to the longitudinal girder.

Transverse stiffness is provided by knee-braces of two angles fastened to the column webs, and to the lower flanges of cross-girders, each column being braced in this way. The ends of pairs of girders resting in the expansion pockets are connected by a light bracing frame.

Truss Spans.

At the Kaw River are used two truss spans, each 300 ft. 10½ in. from center to center of end pins. They are of the double-intersection, deck, Warren type, with subdivided panels, there being ten main panels. The depth from center to center of chords is 37 ft. 6 in., and the trusses are spaced 33 ft. apart. These spans are riveted throughout, and attention is called to their exceptional size; each span weighs about 900 tons.

The floor system is arranged just as on the trestle, with cross-beams and cantilevers riveted into the top chords and short vertical posts. Two small expansion joints for the floor are used on each span to avoid the distortion of the floor-beams as the top chord shortens.

The top chords are of two web-plates, a cover-plate, and four angles, with the upper angles placed inside. The various top chords are spliced only enough to assure alignment, and the contact of abutting webs is relied on to carry the compression. The bottom chords are of two built channels laced with angles. The mid-span chord has an area of 212 sq. in.

Diagonals are alternately of built channels turned out, and of built I-struts, the latter passing through the former. Thus the channels rivet on the outside of the gusset-plates and the I's between them.

The upper laterals are of two-angle diagonals placed in the plane of the lower edge of the chord. They reach the chords only at main panel points, and therefore intersect at the intermediate floor-beams, to which they are attached by plates passing through slotted holes in the webs, and are connected thereto by lug-angles. The lower laterals consist of diagonals, between main panel points, of laced, four-angle, I-struts, the depth of the chords.

The upper laterals are made heavy enough to dispense with inter-

mediate transverse bracing between the trusses. Between the end posts there is cross-bracing of four-angle I-struts from the floor-beam above to a transverse strut below. This arrangement avoids the transference of loads from truss to truss due to unequal deflections, and also the unsatisfactory details of transverse bracing fastened to other than vertical members.

The shoes are built up of plates and angles, and are connected to the trusses by 12-in. pins. Ten segmental rollers, 8 in. high, support the expansion shoes, and cast-steel base-plates are used at each end.

The trusses were calculated on the assumption of all loads being equally distributed between the two systems. Although the trusses are equally loaded when fully loaded, the resultant stresses vary somewhat, because the larger part of the load on the highway truss is from the concrete floor, while on the motorway truss the live load is much greater. Both were calculated, and, in the make-up of sections, the chord stresses of the highway truss were combined with the web stresses of the railway truss and the two trusses were made identical.

The one other truss bridge of the viaduct is a skewed span, 147 ft. long, over railway tracks. This span is of the same general type as that at the Kaw River, except that the intermediate vertical posts are absent, the floor-beams being supported on the gusset-plates. The depth of trusses is 15 ft. 6 in. At each end, owing to the skew, there is a diagonal girder, 62 ft. long, and the regular cross-beams are riveted into it. The span is supported on shoes of the usual type, that at the roller end resting on seven segmental rollers supported on a cast-steel base. There is no bracing, either longitudinal or transverse, in the columns under this span, and especially heavy sections were used. The columns supporting the fixed end are 42 by 32 in., having about 200 sq. in. of section. Each of the four columns is anchored to its concrete pedestal by eight 2¼-in. bolts 9 ft. long.

Specifications and Manufacture.—The consulting engineers prepared complete detailed drawings, and from these the contractor prepared working drawings. These engineers' drawings do not give in full all minor details, such as the exact spacing of rivets, sizes of small plates, etc., but do fix the dimensions of all members and of all parts thereof, allowing the shops to fill in the lacking dimensions according to their preference. The shop drawings were then checked by the consulting engineers, who made the necessary corrections, and finally approved all details. As a representative, located at the shops,

gave all points prompt attention and decision, much time was saved and correspondence avoided.

All metal is open-hearth, medium steel. The specifications provided that all plates should be rolled from slabs of at least six times the plate thickness, which slabs were to be made by rolling an ingot and cutting off the scrap. The following maximum percentages of ingredients were fixed:

Phosphorus, 0.04 to 0.06; sulphur, 0.04; silicon, 0.04; manganese, 0.70. The ultimate tensile strength on test pieces was to be from 60 000 to 70 000 lb., and the least allowable elastic limit, 35 000 lb. The least allowable elongations on specimens varied from 20 to 24%, with a corresponding reduction of area of from 44 to 36 per cent.

First-class workmanship in every respect was specified. Sheared or hot-cut edges of plates were planed so as to remove $\frac{1}{4}$ in. of metal. Rivet holes were sub-punched $\frac{1}{8}$ in. less, and reamed to a diameter $\frac{1}{16}$ in. greater than that of the rivet, and it was required that the reamed holes be perpendicular to the metal surface. A modification in part of the foregoing was made, and the field holes for the stringer bracing and certain other minor parts were punched full size.

The ends of all girders and abutting members were planed to make perfect contact. Stiffeners on girders were ground so as to fit tightly against the flanges. This was strictly enforced in the case of stiffeners carrying concentrated loads, and in certain cases where the stiffeners did not quite touch they were removed and new angles riveted on. Edges of spliced web plates were planed and put in contact for their full length.

It was specified that special attention be given to the cleaning and painting of the metal-work in the shop, where one coat of paint was applied. The metal was cleaned of rust, dirt, and loose scale before painting, and all surfaces of metal in contact were required to be painted before being put together. In a few instances it was necessary to have finished work cut apart in order to do this painting.

A large portion of the work was laid out direct, without the use of templates. Gauge punches were used almost exclusively on pieces which duplicated several times. There was much duplication, of which could be mentioned 220 45-ft. girders, 480 floor-beams, 350 cantilever-beams, practically all stringers, and so forth. Although the columns are all of different lengths, groups were made together requiring only the details near the lower ends to be varied.

The rate of shop work varied considerably, but averaged about 900 tons per month, finished and shipped. The Kaw Bridge spans, comprising some 1 800 tons, were fabricated and shipped in five weeks.

Careful inspection was made of all operations in the shop. Rivets were examined as driven; there was a general supervision of laying out, punching, shearing, assembling, etc.; and the finished piece was checked before loading. The result of this careful inspection was apparent in the very few errors found in the field.

Full records of the progress of each piece were kept, and weekly reports were made giving the status of work of all members and the percentage of the estimated total completed. The weights were all calculated from the drawings, and checked against the scale weights.

2.—Design and Construction of Substructure.

Surveys and borings showed bed-rock to be about 50 ft. below the ground surface across the entire valley except for a few hundred feet from each bluff, where it is very much lower. The material overlying bed-rock was found to vary from soft muck and running sand to clay, firm sand, and gravel.

Because of the uncertain bearing capacity of this accreted land, and of a possibility of scour, piles were used under 276 of the 326 pedestals of the viaduct.

The first design contemplated the use throughout of a patented form of concrete pile, and the pedestals were designed accordingly. This patented pile was to have been built by driving into the ground a steel shell or mould having a removable point, and on reaching the required depth the shell was to be filled with concrete and gradually pulled out as the concrete was tamped into place. Further consideration and examination of driven piles made it seem to be undesirable and inexpedient to use this patented pile in the wet soil to be encountered.

The manifest desirability of a concrete pile made the engineers loath to abandon that type of foundation, and led to the adoption of a built-up driven pile. A number of experimental built-up piles were made at once, and, after the feasibility of driving them was demonstrated, their general construction was immediately undertaken.

Of the pedestals not supported on piles, some rest on rock, some on hard clay, and some on soft material. Thirteen pedestals near the east end of the viaduct are under the buildings of a machine shop or under

railroad tracks. The material here is very soft, and, after experiments, an intensity on the soil of 1 ton per sq. ft. was allowed. In order to have the excavations shallow, the bases were reinforced. Plain steel bars were used.

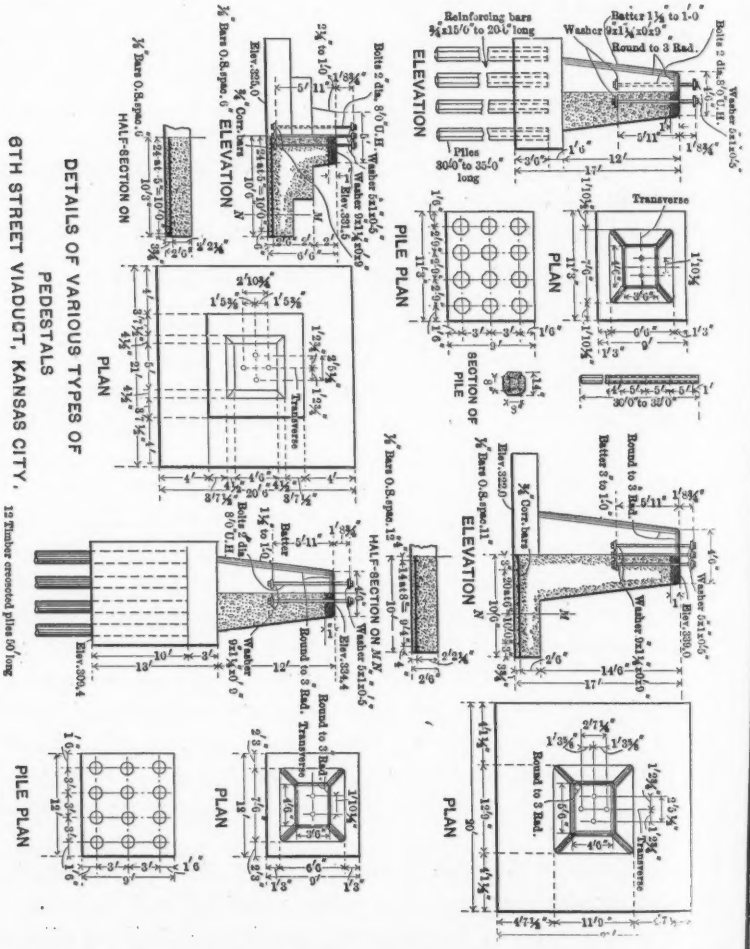


FIG. 4.

Near the western end of the viaduct the pedestals rest on firm red clay, loading it to about 3 tons per sq. ft. The three Kaw River piers are founded on rock, two being sunk by the pneumatic process, and one built by open excavation.

FIG. 1.—CONCRETE PILES: FINISHED PILES CURING.

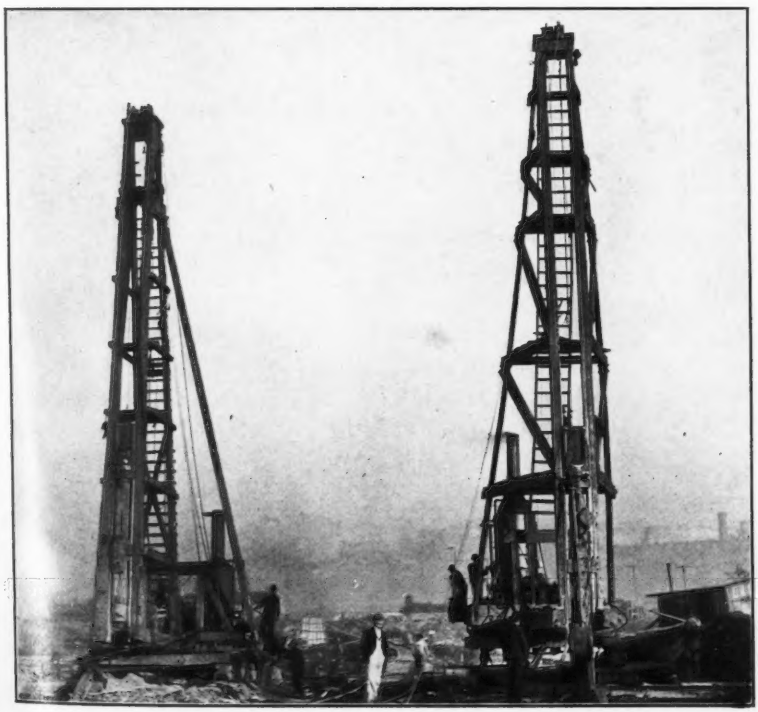


FIG. 2.—PILE-DRIVERS, SHOWING SPECIAL ARRANGEMENT OF FOLLOWERS.

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Piles.—The moulded concrete piles are all of one size, 30 ft. long, 10 in. square at the lower, and 16 in. square at the upper end, with a diagonal 3-in. cut off each corner. Each is reinforced by four ⅞-in. steel bars, extending from end to end, symmetrically placed as near the surface as practicable, and surrounded by a cylinder of heavy woven-wire fencing intended to occupy a position about 1½ in. from the surface of the concrete. In the later piles the wire was omitted.

The forms or moulds for the piles were simple boxes, with bottom, sides, and ends of 2-in. plank suitably cross-braced. Lugs on the cross-braces engaged wooden wedges which tightened and held the parts together yet readily permitted them to be knocked down for moving. The diagonal corner moulding made the form almost water-tight. The upper end of the pile was shaped into a round head to facilitate driving, and the lower end was bluntly pointed. The several parts, of course, were made interchangeable. The interior surfaces were coated with oil before concreting. The piles were moulded lying on the ground, in groups of from 8 to 16, in position convenient to their point of driving.

The specifications provided that the piles be made of concrete composed of "1 part of Portland cement, 3 parts of Kaw River sand, and 5 parts of clean hard broken stone to pass a 1-in. iron ring". The cement used was Iola Portland, and cost, f. o. b. Kansas City, $1.60 per barrel. The sand of the Kaw River is very clean, sharp, and well graded, testing about 52% retained on a No. 30 sieve; its cost, delivered, was 75 cents per cu. yd. The stone was from quarries about the city, and cost $1.30 per cu. yd. delivered. There was considerable trouble in securing clean stone, for the limestone strata of this region are overlaid with clay, and, especially in a rainy season, it is impossible to prevent some clay from intermingling with the stone in the crusher bins.

Some limited experiments were undertaken to determine whether the amount of clay present had a deleterious effect on the concrete. Test bars made of measured quantities of material, mixed in a uniform manner, were prepared with the stone as it was delivered from the crusher, and with stone carefully and thoroughly washed and cleaned. When these bars were tested and compared it was found in all cases that the unwashed clayey stone made stronger concrete than the clean washed stone.

The concrete for the piles was made by hand on tight movable platforms. The practice was to mix together the stone, sand, and cement dry, and then add enough water to give a reasonably wet mixture. The material was distributed by wheel-barrows, and dumped into the prepared forms. After a 2-in. layer of concrete had been spread in the form, the reinforcing of wire and rods, already prepared, was laid in position, the remainder of the concrete was dumped in, tamped to place, and smoothed on top to a reasonably even surface. Sticks were used to tamp through the meshes of the wire and to hold the rods apart. The rods had a tendency to bunch together, and it required constant care and watchfulness to get them where they belonged. Several methods were tried to obtain the desired results easily, but none was entirely satisfactory. The conclusion is, however, that stiff iron spacers, either as a ring or as an X, at intervals of about 3 ft. along the rods, would give superior results.

Each pile was marked with the date made, and for several days thereafter was showered copiously with water. The side and end forms were usually removed in 48 hours, but the pile was not rolled from its base for about a week.

Over-anxiety to re-use the base forms without delay caused the loss of several piles, for the heavy green piles were easily cracked by very small inequalities of the ground. Weather conditions, of course, affected the rapidity with which the piles hardened, but they were generally allowed to cure from 3 to 5 weeks before use, although some were driven 2 weeks after being made. Each pile contains 1.4 cu. yd. of concrete. The finished piles are shown in Fig. 1, Plate III.

For driving, or rather sinking, the piles, heavy turn-table drivers, equipped with a special kind of follower, were used. This follower was of timber, about 12 ft. long, with guides to fit the leads at its upper end and mid-point, and with a deep cylindrical collar at its lower end. A standard 3 000-lb. hammer rested on the follower and was fastened loosely to it. The usual "pile line" was carried down, through a steel sheave, then up and made fast to the top of the leads. To this sheave was fastened a 10-ft. length of steel cable having a slip-noose at its lower end, and a "pull-off line" attached to the slip-loop of the noose. A water-jet pipe, consisting of a 2½-in. pipe 35 ft. long, with its lower end drawn down to a nozzle, completed the equipment. This jet pipe was handled by a line to a spool of the engine, and was supplied with

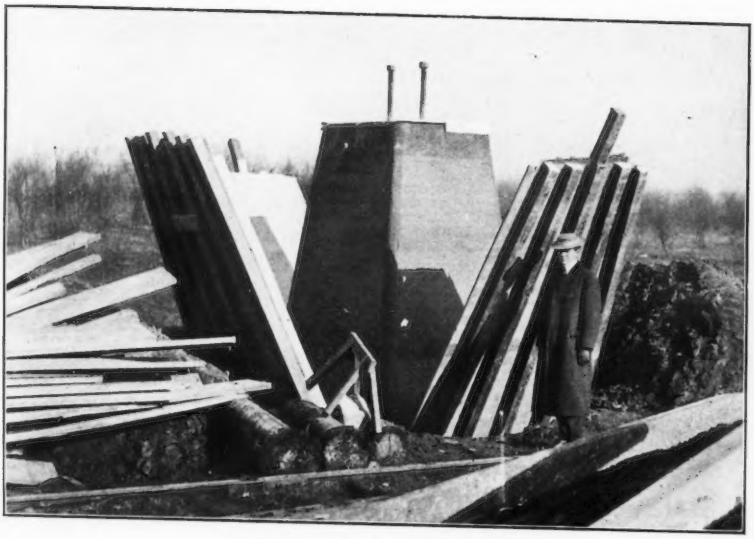

FIG. 1.—PEDESTALS: FORM REMOVED.

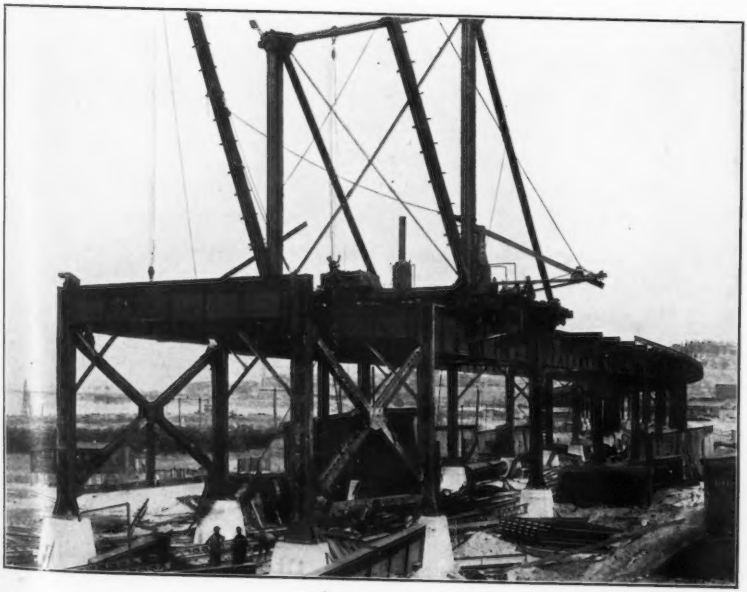

FIG. 2.—TOWER SPAN, WITH LONGITUDINAL BRACING.

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water at a pressure of about 120 lb. through a 3-in. standard fire-hose. To provide the water for the jets, a pumping plant, consisting of a high-duty 18 by 10 by 12-in. fire pump with suitable boiler equipment, was erected on the bank of the Missouri River about $\frac{1}{8}$ mile from the structure, and a pipe line, from 3 to 4 in. in diameter, was laid from it along the entire viaduct. Fig. 2, Plate III, shows the pile-drivers and the special arrangements of the followers.

In general, piles were driven before any excavating was done, and they were sunk so that their upper ends were from 1 to 5 ft. below the ground surface, in the following manner:

After the driver had been set to the correct position by the adjustment of the rollers on which it stood, the hammer and follower were lifted to the top of the leads. The jet was then played on the desired position of the pile, and, as the ground softened, it was gradually pushed down full length. In the meantime the slip-noose cable was placed around a pile some 4 ft. from its upper end and the pile was raised and swung into the leads where it was straightened up and the follower settled carefully on the protruding head. By means of the sheave-line, the pile was then churned up and down, the pile, the timber follower, and the hammer resting thereon all going up and down together, while the jet was kept in constant service. Drops of as much as 6 or 7 ft. were made, without apparent injury to the pile, and the churning process was continued until the required depth was secured. By the "pull-off" cable, the slip-noose was easily loosened and removed.

The effect of running the jet down first was to cause the pile to keep its correct position, as it was found that otherwise each pile had a tendency to travel toward the adjacent ones previously driven, where the ground was still soft; and even then the pile tended to lean toward the side where the jet was played during sinking. It is believed that more accurate sinking could have been done with a pair of jets, one on each side. The simple slip-noose for handling the piles was developed only after the failure of several more or less elaborate schemes devised for the same purpose.

The time consumed in the actual sinking of a pile varied somewhat, but, where no particular obstructions were encountered, from 2 to 5 min. sufficed. However, the highest record for one gang in 10 hours was 36 piles.

It was soon apparent that, owing to the delay in starting, cold

weather would probably set in and stop concreting before all the concrete piles could be made, and it was necessary to devise a substitute so that the structure might be finished in the time assigned. Creosoted pine piles were, with reluctancy, decided upon, a rush order was given to pile contractors and creosoting plants, and, by constant attention, the piles were rapidly delivered.

The specifications called for:

"Piles cut from live, straight, long-leaf, yellow-pine timber; to be free from cracks or wind-shakes; to be so straight as to be at no point more than one-third of their sectional diameter out of a straight line joining the centers of the ends; to be of even, gradual taper from end to end; and to be not less than 10 in. in diameter at the small end."

The specifications for creosoting provided, after the usual requirements for dressing, peeling, and artificial seasoning, that:

"After the seasoning is entirely finished, pure, unadulterated dead oil of coal-tar shall be injected into the timber so that to each cubic foot of timber will be 12 lb. of creosote oil."

Additional equipment for handling the timber piles was hastily provided. The drivers were of the ordinary light timber-framed type, braced by guy lines to the top of the leads, which were 60 ft. high. Each was mounted on two wooden rollers, 1 ft. in diameter and 20 ft. long, resting on timber sills laid on the ground. Lateral movement was secured by sliding the driver along the rollers. This roller arrangement proved good, permitting rapid moving and setting. Both drop-hammers and steam-hammers were used, the latter doing much more efficient work. Water-jets, arranged about as previously described, were used. The efficiency of the jets was demonstrated in several instances when, because of a break-down of the pumps, attempts were made to drive without water, for these invariably resulted in the piles being injured and broomed, before they had reached their full depth, thus precluding further driving.

On a portion of the structure, 50-ft. piles were driven, reaching to bed-rock; on the remainder, 35-ft. piles were used, resting, as do the concrete piles, on gravel and on clay.

To determine the bearing capacity of a concrete pile, a test load was placed on one of the first piles sunk. The pile loaded was of standard size, was made on August 28th, driven on September 15th in 25 min., and the loading was begun on September 24th. To support

the load, a timber platform, 12 by 4 ft., was balanced on a single post resting on the pile head. Observations were made with an engineer's level, reading on a scale fastened to a piece of 1-in. pipe which was set into the pile a few inches and projected above the platform without being in contact with it. The load was of pig iron, each pig weighing about 66 lb., and was placed by hand at an average rate of 6 tons per hour.

During the first 1¾ hours the load increased from 4 tons (the weight of the platform) to 19 tons, with a very small settlement. This load was constant for the next 15½ hours, and the pile settled 0.012 ft. During the next 4 hours the load increased from 19 to 35 tons, with an additional settlement of 0.038 ft. After an interval of 1½ hours, loading was resumed for 4 hours and 40 min., with a settlement of 0.097 ft. During the 17 hours and 40 min. this total load was kept on the pile, there was a further settlement of 0.062 ft. In the 4½ hours of unloading, the load on the pile was reduced from 43.75 to 4 tons, and the pile rose 0.021 ft., leaving a net settlement of 0.198 ft.

These results made necessary some revision in the pedestal design, as very much greater capacity had been claimed for the patented pile. Moreover, the material varied so much from place to place that this test was not believed to be representative of the entire structure, and some ready method for tests was desirable. It was clearly impracticable to load each pile to determine its bearing capacity, and as there is no way to estimate the capacity of piles sunk by jet, the following plan was adopted.

Creosoted pine piles, 40 ft. long, were furnished and called "test piles." In cases where, by reason of special softness of the ground in driving adjacent pedestals, or from the data of the borings, there was uncertainty, one or two of these "test piles" were substituted for the concrete piles. These were driven by using the drop-hammer in the ordinary way, without a jet, and before any jetting had been done in the immediate vicinity. Observations were made of the drop of the hammer and the penetration of the pile from 25 to 30 ft. down, or about the position to be occupied by the concrete piles, and the resultant bearing capacity was calculated by the well-known formulas. In all cases these results gave a greater safe load than that actually on the piles, in fact, it was usually necessary to resort to the jet to get the timber piles to the full depth.

In the main viaduct there are 798 concrete piles, a total of 23 950 lin. ft. in place; and 2 573 creosoted timber piles, aggregating 93 800 lin. ft. in place; a total of all piles of 117 750 lin. ft.

Excavation and Concreting.—The excavation for pedestal bases was comparatively simple, and the work was done by hand. In nearly all instances where concrete and the shorter timber piles were used, no sheeting or bracing was needed, the sandy, clayey material standing to depths of 8 or 9 ft. For about 80 pedestals, most of which required pits some 13 ft. deep in very boggy, sticky ground, sheet-piles of 2-in. plank were driven by hand around timber frames, and these frames were dropped down to brace the planks as the excavation proceeded.

As soon as the piles were sufficiently exposed they were sawn off at the required elevation. Before concreting they were cleaned of mud and filth by scraping with a hoe having a curved blade. Where concrete piles were not settled to full depth they were cut off with chisels, and the reinforcing bars bent over.

A number of pedestals, particularly those at the east end, under buildings and near tracks, required special attention in excavation with regard to bracing and shoring. One building, under which several were constructed, having been through two fires and a flood, was in a precarious condition, especially as it rested on soft flowing material. The base of one pedestal under it extends under the bed of an engine which was kept in operation by supporting it temporarily until the concreting was completed. For certain railway tracks carrying a heavy and frequent traffic, it was necessary to drive temporary trestles to permit the construction of adjacent pedestals.

Forms.—In most cases no forms were used for the bases of pedestals, the pits being merely filled with concrete to the required elevation. The pedestal shafts are practically of uniform dimensions, and the forms were devised for repeated use. Each side was built as a single piece, of 1-in. horizontal, matched, dressed boards, with vertical posts of 6-in. square timbers. The four sides were held together by three square frames or hoops of 3-in. plank, bolted at each corner, bearing sidewise against the studs. Wedges were driven behind all posts not properly bearing. These shaft forms were easily adjusted to correct position by the insertion of small wedges under the bottom. Sheet-iron corner moulds were used to give the concrete a curved corner of 3 in. radius. For the vertical edges, probably the best results were secured

by using sheet iron of No. 12 or 14 gauge, rolled to approximate radius and then nailed firmly to one side of the form and tacked to the adjacent side only enough to keep the concrete from getting behind. No particular harm was done to this corner mould in pulling down the form. The upper corners were shaped by sheet-iron or wooden moulds, or by the use of a curved trowel. The latter, in the hands of a skilled workman, gave excellent results, but its use was deprecated because of apparent desire to use unskilled labor. Two wooden templates for supporting and holding in place the anchor-bolts, all of which were placed as the concreting proceeded, were fastened to the form.

Concrete.—The concrete was specified to be mixed by machinery in the proportions: One part of Portland cement, three parts of clean, coarse, sharp sand, and five parts of hard, clean, broken stone to pass a 2½-in. iron ring; the proportions to be determined by volume; all ingredients to be measured loose; and enough water to be used to form a wet concrete.

In addition to the usual requirements for package delivery, etc., the specifications for the cement were: To be so fine that 97% in weight will pass a No. 74 sieve, and 90% pass a No. 100 sieve; to be of such tensile strength that briquettes of neat cement will show, for 1 day, from 150 to 250 lb. per sq. in.; for 28 days, from 400 to 600 lb. per sq. in.; to show no drop in strength; to be slow-setting; preliminary set in not less than 30 min., final set in not less than 3 hours; to undergo boiling and steaming tests without failure.

Sand was defined as "particles of hard, clean stone which will pass a ¼-in. mesh sieve, and not less than 50% of which shall be retained on a No. 30 sieve." Both the Kaw and Missouri Rivers have great supplies of excellent sand, that of the former being somewhat cleaner, but both coming within the requirements.

"Sound, hard rock, free from all dust and dirt," was called for, but, as has been noted, the output of local crushers, containing more or less clay, was used. It is difficult to state just how much clay was allowed to be in the stone, as a given load would seem very muddy when wet, yet clean when dry. The stone was generally accepted when there were no lumps of clay, or of clay and stone chips, even though many of the stones were muddy. Inspectors were stationed at the quarries to examine the material as loaded, yet, in spite of all care, some stone was used which contained small balls of clay.

Special detailed specifications in regard to mixing concrete were distributed among the various sub-contractors, to assure uniformity of product.

The concrete was mixed so wet that no real "tamping" could be done, but it was deposited approximately in layers of from 9 to 15 in., and "thoroughly spaded" by thrusting a spade to the full depth of the blade at intervals over the entire area, special attention being given to spading near the surfaces of piles and forms, so that the mortar would have perfect contact. It was in general handled from the mixers in wheel-barrows, either on the ground or on elevated runs, as conditions required, and was thrown freely from the tops of the pits or forms. For other pedestals and for certain abutments, slightly more equipment was needed, and derricks with boxes of simple well-known types were used in handling materials. Anchor-bolts were usually placed after the lower half of the shaft of the pedestal had set, and they were very carefully adjusted to exact position. The upper foot of each pedestal was of rich concrete made with Joplin flint or granite chips. One of the pedestals, with the form removed, is shown in Fig. 1, Plate IV. The pedestals were finished from $\frac{1}{2}$ to $\frac{3}{4}$ in. low, to allow for grouting under the shoes.

Various types of mechanical concrete mixers were used, both batch and continuous machines. The continuous mixers automatically measured the sand, stone, and cement by means of worms, and were mounted so as to be easily portable. Although considerable pedestal concrete was made with these machines, the results were not as satisfactory as those produced by rotary batch mixers which mixed the concrete in revolving drums or boxes. The output of the continuous mixers was not uniform, even though they measured the materials accurately, especially where the vicissitudes of the work necessitated intermittent operation, with frequent stops and starts; the water was not easily regulated, and the mixture was not homogeneous. Moreover, after a time, numerous spots appeared on the surface of the concrete from these machines. It was found that these were caused by small balls of clay, from $\frac{1}{8}$ to $\frac{3}{4}$ in. diameter, embedded in the concrete, the results, of course, of clayey stone. Careful examination, however, proved that these clay balls were practically absent in the work of the batch mixers, showing that the churning and tumbling had been sufficiently vigorous to pulverize and distribute all lumps of clay, with consequent harmless-

ness; while the mixing of the continuous machines had not been sufficiently vigorous to do so and therefore was inferior.

Certain pedestals, abutments, and retaining walls had special forms, made of 2 by 6-in. matched, dressed flooring, laid on vertical studs spaced from 2 to 3½ ft. from center to center. One side of each form was braced to the ground, and the other side was adjusted to it by numerous malleable-wire ties. A strut, fixing the correct width, was placed near each tie-wire.

Vertical cleavage planes were made in abutments and retaining walls, separating those parts of the wall having different load intensities on the foundation. A cross-partition was built in the form, and the concrete was finished against it. Before building the adjoining part of the wall, the partition was removed, and tar roofing paper was nailed securely to the exposed surface, thus effectually preventing cohesion of the fresh material to the old.

Kaw River Piers.—The three piers of the Kaw River Bridge are of concrete, of the same materials and proportions as used for the pedestals.

The west shore pier rests on a ledge of rock about at low-water level, so that the construction was very simple, and the work was done in the dry. Light sheet-piling was driven around the pit, which was excavated by hand. Hand-pumps were sufficient to take away the seepage water, and the bed-rock was easily exposed, so that its surface could be cleaned, washed and prepared for the concrete. The base of the pier was built up nearly to the ground surface.

The other two piers also rest on bed-rock, and both were sunk by using pneumatic caissons. The caissons were similar; that of the channel pier was built on shore and floated to place, and that of the east shore pier was built in place.

The caissons were of timber and had steel cutting edges. Timbers, 12 in. square, of Oregon fir and Southern pine were used; and were thoroughly bolted and drifted together. The walls of the chamber were 3 ft. thick, and the roof was 6 ft. thick. Every other stick of the upper course of roof timbers was omitted, to give bond for the concrete.

The outer timbers were vertical, and each was fastened by at least two bolts extending into the chamber, besides drifts. Horizontal timbers were lapped at the corners, log-house fashion, thus avoiding extensive framing. The chamber was sheeted inside with 3-in. planks,

beveled for caulking, and fastened with boat spikes driven through bored holes. Every bolt, and the shank of every spike and nail projecting into the chamber was carefully wrapped with oakum before being driven entirely home or finally tightened up. On the outside all bolt heads were countersunk, thus leaving a smooth surface. Two cross-bracing trusses, with adjustable tie-rods, stiffened the walls of the chamber.

A false bottom was put in the caisson for the channel pier, in order to float it to position; and, for this pier, a light tramway, on two pile bents supporting a standard gauge track, was built from shore to pier site for conveying materials. Concrete was mixed on shore and carried out on push-cars in skip boxes. A derrick barge handled the boxes and the timbers for the crib.

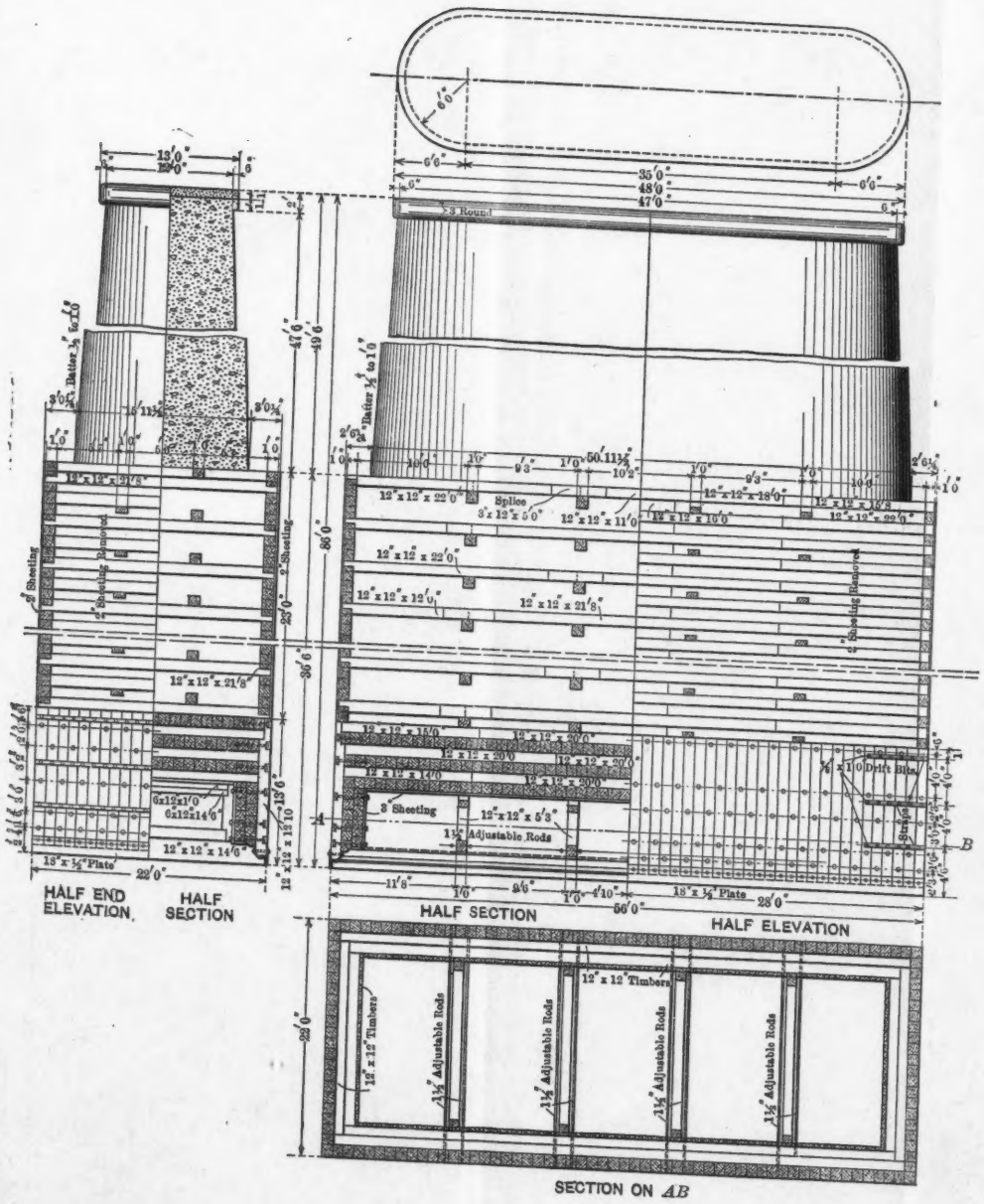
The crib, above the caisson proper, was of one thickness of 12-in. timbers sheeted outside with 2-in. plank nailed on vertically.

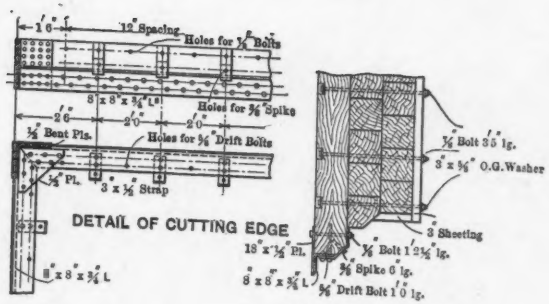
The usual man and material shafts, with air pipes, blow-out pipes, etc., were installed. The locks used were very simple, being made by merely putting cast-iron diaphrams with drop-doors at each end of a section of ordinary shaft. The one objection to locks of this kind is the necessity of an inside lock tender, and this is not entirely a fault, as men are much less likely to be "plugged" by a lock tender with them in the lock than by a man outside who operates without definite knowledge of conditions inside.

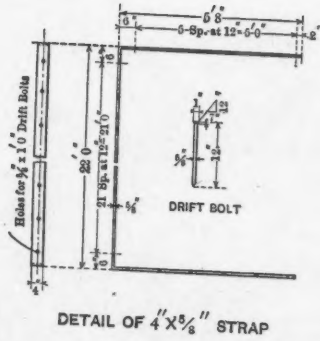
The material excavated was sand, with occasional deposits of silt, and very many boulders, so that most of the material was blown out dry. Numerous relics of the great flood were found in the river bed, indicating deep scour. The metal, notably some street car rails and some parts of steel bridges, was carried down and embedded in the concrete.

Considerable blasting was necessary to prepare the bed-rock for piers. A ditch 3 ft. wide was blasted out around the entire perimeter of the cutting edge, so that the concrete is at no place less than 2.5 ft. below the bed-rock surface. The middle portion of the rock was not removed, but was washed clean, as also was the entire inside of the chamber.

Concrete for filling the chamber was of standard proportions, and was mixed outside and locked into the pier through material shafts 2 ft. in diameter. It was placed in the dry, and carefully tamped to place,







DETAIL OF 4" X ⅝" STRAP

DETAILS OF PIER 2, KAW RIVER BRIDGE
6TH STREET VIADUCT, KANSAS CITY.

UNIV. OF MICH.

especially against the ceiling, the boulders and pebbles carried down being mixed in. When the remaining space had been reduced to such small dimensions that the men could no longer work, the air was cut off, all doors were allowed to swing open, and into all shafts a thin grout was quickly poured from above until it was some feet above their lower ends. Ordinary concrete was deposited to fill them.

Coffer-dams were not used, but the shafts were built up as soon as the crib was of the estimated required height, and from this point down the sinking was handled very carefully so as to keep the shafts in correct position. For the last 16 ft. of sinking for Caisson No. 2 the shaft moved out of position only ½ in. at each end, and that in opposite directions.

The sinking of Caisson No. 1 was complicated by the presence of a 30-in. high-service water main lying so close to the pier site that the caisson had to be rotated to clear it. Pile bents were driven under the pipe for a support, but these went down, and a sling was put around it and passed up over an **A**-frame to a distant anchorage. All attempts to rotate this caisson to its correct position after it was below the obstruction failed.

These caissons were models in construction; they were well lighted by electricity, and constant and adequate air pressure was maintained. Natural gas was available for fuel.

For Pier 3 the shaft form was of 2 by 6-in. matched dressed flooring, which was placed horizontally on the sides of the pier and fastened to vertical studs, but on the curved ends it was placed vertically and drawn into position by §-in. round iron hoops. These hoops were passed through holes in the vertical studs at tangential points and drawn tight by nuts threaded on their ends. Interior circular templates were placed near the bottom and at the top of each 10-ft. section of the end staves to hold them to position, but were removed as the concrete was built up. Ties of heavy malleable wire were put across between pairs of studs, and twisted tight to bring the sides to proper place. These ties were placed with such frequency as to hamper the placing of concrete, but could not be dispensed with. The building of a 10-ft. section of form and of filling it with concrete alternated, so that the pier was stripped below as it was built up above. Forms of this kind are not considered entirely satisfactory, being easily warped and distorted, and often delaying the concreting for readjustment.

The forms for the other two piers were made of the same planking, in 8-ft. lengths, but this was placed vertically all around the pier. Rigid frames, 4 in. thick, with curved ends of built-up plank nailed and bolted together, were spaced 4 ft. apart, and supported the sheeting. Five ½-in. tie-rods held the sides of the frame together, and cross-bracing at each end and on each side provided against lateral displacement.

The lower edges of the copings were formed to the required radius by sheet-iron corner moulds, and the upper edges by the manipulation of a curved trowel. The tops of copings were brought to a smooth level finish by floating with mortar before the concrete had set.

3.—Field Engineering.

The field engineering included surveys, pier location, inspection, and general supervision of all construction.

The alignment of the viaduct was governed somewhat by property boundary lines. The structure lies in three tangents, aggregating about 6 900 ft., and three curves, each of about 1 900 ft. radius, and totaling 1 500 ft. in length.

Exact surveys and final measurements were made to locate street crossings, railways, buildings, etc., before much work was done on detailed drawings. Base lines, paralleling the chosen center line tangents were carefully marked on the ground, and the angles at their intersections determined. Points on base lines, about 400 ft. apart, were marked by large wooden hubs driven down flush. In the boggy, quaky ground these were 4 by 6 in. and from 3 to 5 ft. long, but in the firmer ground not so large. A piece of sheet copper was nailed on top of each hub, and the exact point was indicated by the intersection of two scribed lines. Each main hub was carefully referenced by marking two intersecting lines. Exact distances from hub to hub were determined either by direct measurement or by triangulation. About one-third of the entire length was triangulated. For the systems of triangles, suitable base lines, usually intersecting with the main line, were marked and measured. Measurements were made as follows: Between two main hubs, 2 by 4-in. marking stakes were solidly driven down, about 98 ft. apart; between these, at about 6-ft. centers, small supporting stakes were placed. All these were sawn off or driven down to conform to the grade from main hub to main hub. When necessary a narrow ditch was dug to contain the stakes, it

being desired to support the tape on them and not on the ground. A 100-ft. steel tape, which had been compared with a standard to determine its constants, was used. The required tension was secured with a spring balance, and the distance was marked by thrusting into a marking stake a knife blade with its sharp edge at the desired graduation. The knife was left there and measured from in placing a second knife ahead, and so on until the second main hub was reached, and there the tape was read to an approximate thousandth. Thus any number of repetitional measurements could be made without the confusion resulting from making each intermediate point with a non-erasable mark. About six measurements were made for each hub distance. The extremes varied from the mean about 1 in 80 000. The base line at the Kaw River was conveniently measured on the deck of an adjacent railway bridge, and the triangulation points there consisted merely of punch marks on the end floor-beams at fixed shoes.

The measurement of angles was done by a method of repetitions and reverses which eliminated instrumental errors in the mean of each series of readings. Enough readings were taken at each point to make the triangles close within 1.5″. The largest triangulated distance was 1 131 ft., and was determined by two triangle systems, so that the independent calculated lengths checked within $\frac{3}{32}$ in. The difference in the elevation of the two end points was 65 ft.

After all distances had been determined, the station and plus of each hub was calculated, and locations for the pedestals were measured readily from the nearest hub.

For the location of river piers, the true angular distance from each base line was calculated, and a temporary fore-sight was established by a single carefully-turned angle which was then measured exactly. The distance of the temporary fore-sight from the true line was calculated, and the point in the hub was then moved the necessary distance ($\frac{3}{4}$ to $1\frac{1}{2}$ in.). When the two reference lines and the bridge tangent were projected to the piers they intersected within a $\frac{3}{16}$-in. circle.

For pedestals, and for piers where possible, reference lines at right angles to the tangent were marked by several hubs well driven down. The longitudinal center lines of each row of pedestals were also suitably marked at intervals. Elevations along the entire line were determined, and numerous carefully checked bench-marks established.

The inspection of construction embraced every detail, and daily records of the progress of each kind of work were prepared. For instance, not only did an inspector watch the placing of concrete, but a second man watched the entire process of mixing. There was an ample number of men to make inspection of stone, timber, cement, sand, pile-driving, forms, concreting, erection, riveting, painting, paving, decking, etc., as has been noted previously.

4.—Erection of Superstructure Metal.

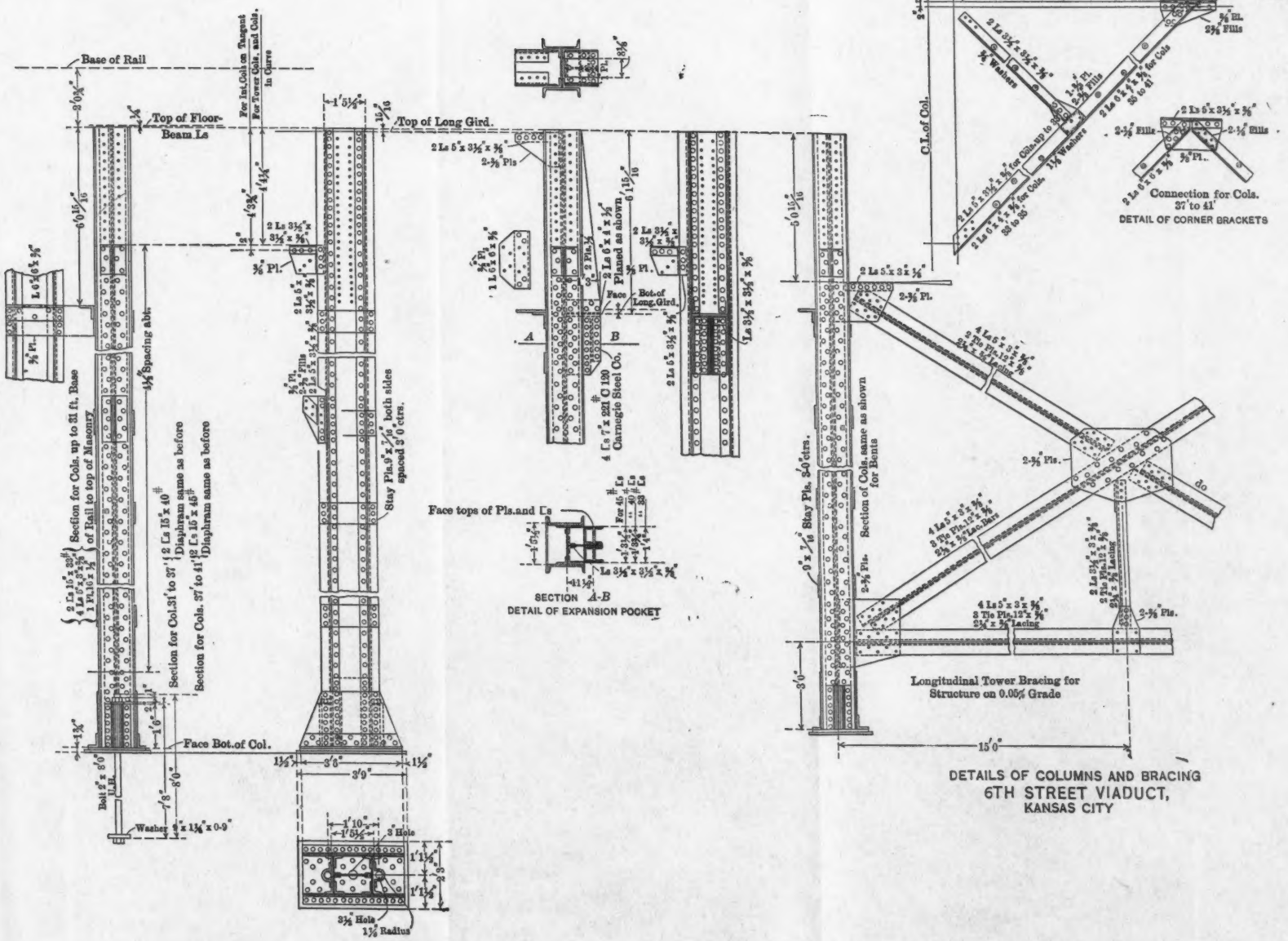
About 88% of the total length of the viaduct is of practically duplicate panels, and, in all details of construction, particularly in the erection, this characteristic feature permitted economic handling of materials and labor.

Trestle Erection.—The traveler used for the ordinary structure consisted of a pair of derricks mounted on a frame arranged to run on track rails laid on the longitudinal girders, the masts thus being 33 ft. from center to center. The details of this traveler are clearly seen on Fig. 2, Plate IV. The masts were double, one stiff and one seated to turn. Each was a 14-in. square timber 40 ft. high, and the booms were built up of spliced timbers to 14 by 26 in., and 60 ft. long. The sills and cross-sills were 8 by 16 in. All connections were made by steel plates, and steel derrick fittings were used throughout. The equipment consisted of two 2-drum, 4-spool hoisting engines of 20 h.p., each handling one boom with its lines and runners. In the ordinary structure the members varied in weight up to 10 tons, which one boom could easily handle.

A service track was laid between the pedestals nearly the entire length of the viaduct, and the steel was unloaded alongside. The intention was to distribute immediately every piece to its proper location, but, owing to the operations of the substructure contractor, this could not be done, and it was necessary to store and redistribute them. The lighter metal was unloaded by a locomotive crane, the larger girders by yard derricks, or by hand upon cribs.

It thus followed that a considerable part was erected from metal already distributed, but also a large part directly from the cars as they were run down from the yard. There was little difference in the rate of erection in the two cases, the former being retarded by the necessity of removing superposed pieces and hitching to material some-

PLATE VI.
PAPERS, AM. SOC. C. E.
FEBRUARY, 1909.
HOWARD ON
THE SIXTH STREET VIADUCT, KANSAS CITY.



DETAILS OF COLUMNS AND BRACING
6TH STREET VIADUCT,
KANSAS CITY

what indiscriminately piled, and the latter by reason of the turbulent operation of service cars over the temporary tracks. Fig. 2, Plate IV.

The pedestals were finished $\frac{1}{2}$ to $\frac{3}{4}$ in. low, and four 2 by 4-in. oak shims of suitable thickness were used to bring each column to the desired elevation. The anchor-bolts had been located so accurately that they sufficed to line up the columns, additional marks being unnecessary.

The usual order of erection per panel was as follows: Two columns were set up at once, one by each boom, and braced up by the anchor-bolts; the connecting floor-beam was placed on its shelf-angles, and bolts were put in one-third of the open holes. The cantilevers were then put up and bolted, after which the main girders were raised singly, both booms being used for the heavier girders. The stringers were then laid on top, to be distributed later, and the traveler was immediately moved forward over the new panel, ready to set the next columns. The maximum day's run was six panels, about 275 tons.

The light material in the deck was put in place in conjunction with the riveting, the stringers by using timber dollies, the bracing, etc., on a push-car running on the flanges of two adjacent stringers. However, as the traveler proceeded, it placed all sway bracing, tower bracing, and laterals.

Certain long and heavy girders required some variation from the usual order of handling. Where the columns ahead were beyond reach of the booms they were raised by a gin-pole or a gallows-frame, which then served to lift one end of the girder, the other end being lifted by the traveler. The longest girder in the structure (107 ft.), weighing 62 tons, was erected in this manner, the gallows-frame consisting of a pair of 18-in. square timbers, 60 ft. high, connected with bracing and caps, equipped with 4-sheave steel blocks reeved with steel cable. Fig. 1, Plate VII.

Because of buildings and railway tracks under the eastern seven panels, the girders could not be handled from below, but it was necessary to run the traveler back, free of obstructions, pick up a girder, set it transversely on the traveler track, move it forward by picking up, booming out, and shifting the traveler, and finally to swing it around longitudinally to its final position. The columns in the buildings were dropped down through holes cut in the roof, with small trouble to the occupants, who, however, did not object to stepping

outside for a few minutes while each girder was being manipulated above their office rooms. The girders immediately adjacent to Bluff Street extend through slots or pockets cut in the retaining wall which supports the street, and rest on pedestals 15 ft. back of, and thus entirely independent of, the wall. The tracks below these girders carry all the passenger traffic into the Union Depot, therefore unusual caution was necessary in erection to guard against possible interruption of train service.

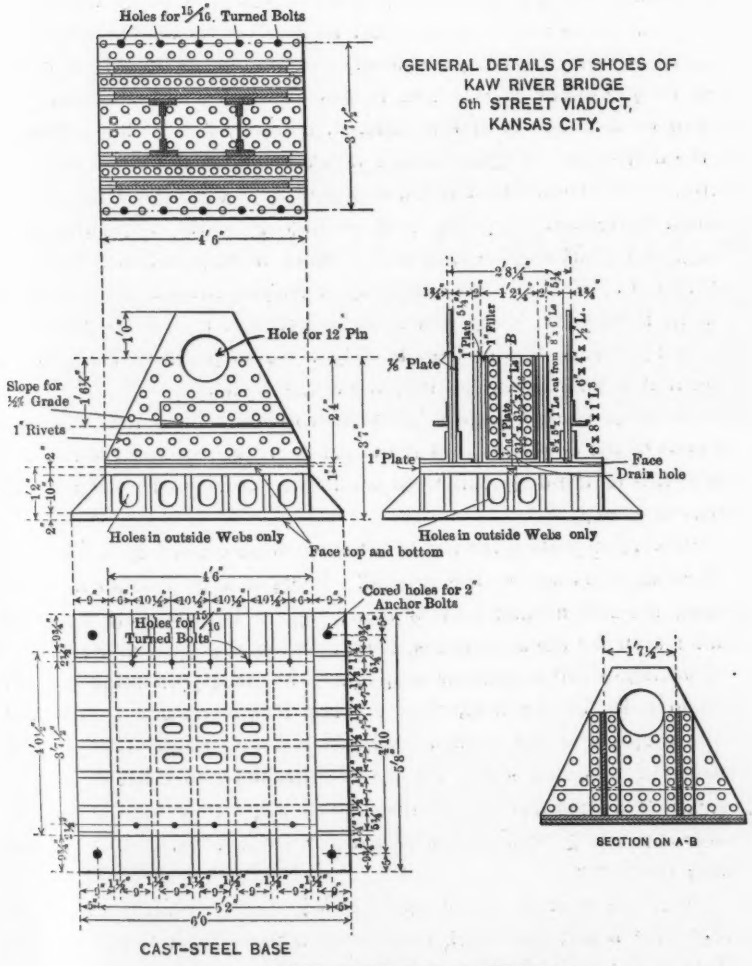
Erection of Truss Bridges.—The falsework consisted of 7-pile bents every 30 ft. under the panel points, and 5-pile bents midway between, the piles being well driven into the river bed. Four 8 by 16-in. stringers were placed under each truss, and a traveler track was supported on two runs of 12 by 12-in. timbers. This traveler was composed of two derricks rigged together, and running on a track between the trusses in the plane of the bottom chords. Beside the traveler, also between the trusses, a standard-gauge track was laid for moving out the metal.

After fastening down the fixed shoes, the bottom chords were laid on the camber blocking for the full length of the span. Beginning at one end, the web members, chords, floor-beams, and laterals, were erected in regular order.

The parts of these trusses are heavy, and the connection plates large and stiff, so that fitting together the various members was tedious work. A clearance of $\frac{1}{16}$ in. was provided, but, in a member 22 in. wide, made of four $\frac{3}{4}$-in. angles and a $\frac{3}{4}$-in. plate, which had to be entered from 5 to 6 ft. into a second member equally stiff, slight irregularities would cause binding. Practically every diagonal and chord section had to be pulled to place with a runner.

It was apparently impossible to do all the field riveting before the time of the expected washout of the falsework, and, in order to support the spans in that contingency, drift-pins were tightly driven into one-third of the open holes, and fitting-up bolts into one-third of the open holes of all main connections. Fortunately, it was not necessary to trust to this expedient, as the expected high water did not come. It is evident the spans would not have fallen, but experience showed that there would have been a slight movement at each point, which, in the aggregate, would have caused trouble.

The trusses were set carefully to correct the camber which was



FIXED SHOE FOR WEST SPAN

FIG. 5.

maintained until the erection was complete. With the passage of the time required for riveting, the falsework settled at points, permitting the entire truss to settle. Attempts were made to jack up the trusses, but the only result was a pushing down of the piles under the wedges. A regular camber of about one-half as much as estimated was finally secured. With the complete dead load and such live load as has been on the spans, the bottom chords are just about straight. The serious consequence of this settlement, however, was in the tightening of the drift-pins. These were not taken out until the rivets had been driven in all the adjacent holes, and then it was found there had been enough movement to pinch them tightly in place, requiring much energy and effort to remove them. Most of these pins had been driven toward the inside, and the cramped quarters prevented efficient striking to back them out. Many had to be drilled out, and, as they were of very hard steel, it was difficult to keep the drill point on the pins and prevent it from running off into the softer metal.

The lower chord splices of these spans were riveted first, and then the ends of the diagonals; but the upper chord splices and the intersection points of the diagonals were not reamed or riveted until the span had been swung.

The similar trusses for the 147-ft. span were erected on a falsework of framed bents set on the ground. There was no trouble in maintaining the full camber in this span; and, as the members are much smaller than for the Kaw spans, their erection was quite simple.

Riveting.—The contractor sought and received permission to ream all field holes in the field, thus making it unnecessary to assemble in the shops; and the metal as shipped out was merely sub-punched. Because of this, and since, for the unrestricted movement of various gangs, it was desirable to keep them separated, a very large amount of scaffolding was placed and retained for some time, but it was of simple character.

The joints were first thoroughly fitted up, and bolts were put in half of the holes; the other holes were then reamed, the bolts were shifted to the reamed holes, and the reaming was completed.

Each joint was thus fitted twice before the rivets were driven, but trials of reaming and then riveting half a joint, and then reaming and riveting the remaining half resulted in the interference of the various gangs, and better progress was made by the procedure given.

FIG. 1.—GALLOWS FRAME USED IN THE ERECTION OF LONG GIRDERS.

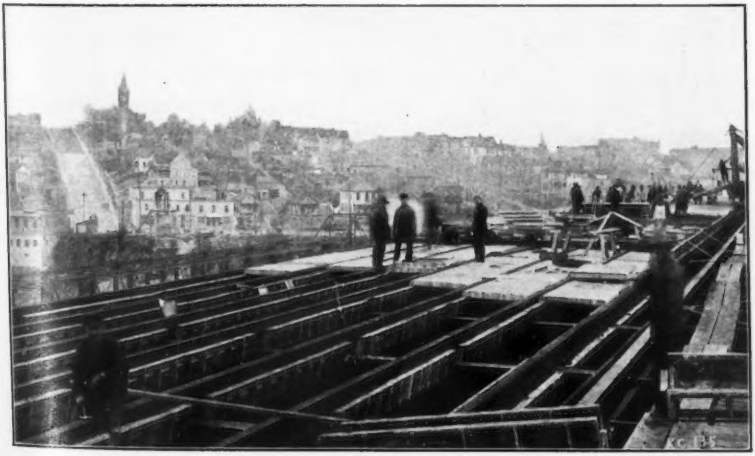


FIG. 2.—CONCRETE FLOOR-TROUGH AND COVER STYLE OF FORM.

Neglect in fitting up caused the majority of cut-out rivets, for if the plates were not drawn tightly together small particles of reamer drillings would work in between them and cause successive rivets to loosen those previously driven. Natural conditions were seized upon to assist this work; for instance, the expansion joints were regularly rammed full of steel wedges, thus gaining the thrust of expansion to force together columns and girders.

The reaming was done with Little Giant, No. 0, pneumatic drills, equipped with 4-flute, tapered reamers having barrels at least as long as the length of the hole. Water, with a little soap, was used as a lubricant, and was kept constantly flowing on the bits. A knuckle-joint was tried for points where the reamer could not be held normal to the metal, but gave indifferent results; so, when unavoidable, holes were reamed on a slight bevel, but they were required to be perpendicular to the direction of stress. There was some trouble in making all holes of the same diameter, for the reamers had to be held very steady to accomplish this. The ordinary run for two men was from 200 to 250 holes, $\frac{7}{8}$ in. in diameter, with a 3-in. grip, or equivalent, in 8 hours. The smooth, clean holes permitted the rapid driving of rivets without further drifting or fitting, and without the delay incident to entering hot rivets into imperfect holes. This was highly essential, especially in such cases as the lower chord splices of the Kaw River spans, where the rivets are 1 in. in diameter and have a 6-in. grip.

Cleveland air hammers, with a 9-in. stroke, operating under a pressure of 110 lb. were used throughout. Grip dollies were required wherever they could be utilized, and hand-forges were used almost exclusively. With one heater and two riveters, 550 rivets, of $\frac{7}{8}$-in. diameter and 3-in. grip, were driven in 8 hours.

The greater part of the riveting was open, easy work, although there were marked exceptions. The rivets in expansion pockets required special, crooked-jamb, joint dollies, in order to pass around the end stiffeners and rest on the next adjacent stiffener.

The bracing of the motorway stringers was connected by $\frac{3}{4}$-in. rivets driven by hand without reaming—in fact, it was found that the lug-angles on the beams were set so high that the rivets were under the flanges, thus precluding the use of an air hammer. Very fair results were achieved, but it would have been much better to have had wider lug-angles.

In the main structure there are about 300 000 field rivets. While accurate costs are not available, the following figures are a close approximation: Reaming cost 7 cents per hole, of which 5 cents was for labor and 2 cents for plant and tools; driving cost 8½ cents per rivet, 8 cents for labor, and ½ cent for plant and tools. It is possible that too much of the plant cost is charged to reaming and too little to riveting. There were reported cut-outs of 3⅓%, but apparently all the rivets were not accounted for, and 5% would be nearer the true figure.

Grouting Columns.—The space below each column base was filled with grout. A cement mortar of ordinary consistency was first prepared and a little daubed under the edges of the base plate around its entire perimeter and finished off neatly. After a day's time there would remain a tight dam all around, pierced, however, by a small hole on each side. The nuts and washers on the anchor-bolts were temporarily removed. Grout, diluted to a pouring thickness, was made in the proportion of about 1 of cement to 3 of fine sand containing a little clay. Several bucketfuls of the mixture were poured, as quickly as possible, into the anchor-bolt holes and the central hole until it ran out of all the "air holes," and then these were promptly daubed up. Mortar was added to fill entirely the side pockets; the foot of each column was filled with concrete finished off with mortar.

This method was highly satisfactory, as it gave a full and even bearing under the entire surface of the base plate, as was proven by lifting up columns which had been grouted.

For large solid castings, as under the Kaw spans, grout could not be used. The bases were set on steel shims about ¾ in. above the concrete. A stop was put at one side, and small quantities of almost dry 1:1 mortar of cement and sand were put under the opposite side, pushed clear across, and rammed with a piece of ½-in. plate. The entire space was filled from one side, thus insuring the absence of voids.

Had holes through the casting been provided, the former method could have been used with superior dispatch and ease.

Painting.—The steel received one coat of paint in the shops, and presented a good even coating when unloaded at the site. As it was erected, all abutting parts and surfaces inaccessible after erection were coated with a heavy paint cement daubed on in generous measure.

Upon the completion of the riveting of a section of the viaduct, the entire surface of the metal was examined, carefully cleaned with wire brush and scraper where necessary, and the heads of field rivets and all areas where the shop coat was scraped off or unduly marred were touched up with paint. Reaming borings added considerably to the ordinary work of cleaning. When these spots had dried, the first field coat (of green paint) was applied. The paint was delivered ready mixed, and was of rather stiff consistency, requiring, for proper application, small amounts thoroughly rubbed on by the vigorous use of a stiff-bristle brush held almost normal to the surface.

When this coat had dried, requiring a varying time, but determined when it became impossible to "rub off" the dried surface skin and expose wet paint below, the final coat was put on. The same care was used in application, and the entire structure was finished up progressively.

The inspectors then examined the work carefully, and remained with the finishers until the painting was acceptable. Small hand-mirrors were used to reflect the sunlight and illuminate dark corners and shadowed areas, thus exposing neglected patches almost impossible to locate by any other means. In many places, too, as, for instance, inside a column, it is much more convenient to observe the reflection in a mirror than to attempt to view the parts directly.

No painting was done in wet, or wet and freezing, weather, but the work was never stopped because of low temperature, provided the surface to be coated was dry and free from sleet or ice.

5.—Viaduct Floors.

The general details of the reinforced concrete floor are shown on Fig. 3. The average slab thickness is 6 in. above the stringers, the stringers being entirely encased, both to protect them from the smoke of locomotives below and to secure the added strength of the filled haunches. The amount of reinforcement is considerably in excess of actual load requirements, in order to provide for the temperature stresses of the concrete. It consists of transverse and longitudinal bars, near both the upper and lower surfaces of the slab, of extra bars over the cross-beams, of numerous vertical bars in the curbs, and of wire netting around each I-beam stringer. To support the hand-rail

posts, bolts project through the sides, the lower bolts passing through the I-beams.

In order that the usual paving could be continued without interruption over the points of expansion, a flexible connection was devised consisting of U-shaped plates bolted side by side. Thus a sort of bellows trough is formed, about 6 in. high and 2 ft. wide. One such bellows extends across the roadway at each expansion joint of the stringers, and is riveted to each stringer on both sides of the opening. The plates are thin enough to spring easily under the movement of the steel. The concrete is finished up to each side of the bellows, and fastened to it by bolts. These troughs were to be filled with a rather soft asphaltic material, the flow of which would accommodate it to the varying position of the containing plates.

The specifications provide for concrete made of cement to be supplied by the company; of clean, hard, broken stone "free from all dust, dirt, shale, rotten stone, or other objectionable material", to pass a 1-in. ring; and of Kaw River sand in the proportions 1 : 3 : 5 by volume, loose. To provide a bond for the contemplated wearing surface of asphaltum (which was to be laid without the usual binder course) it was required that clean stones be sprinkled on the surface of the wet concrete, and be tamped into it enough to embed them partially, yet permit them to project a little. Other principal specifications were: That the entire thickness of the floor was to be deposited in one continuous operation so as to avoid laminations or dry horizontal planes between consecutive layers; that all surfaces were to be finished neatly to the exact required dimensions; that all bolts and embedded castings were to be set to exact position; that reinforcing bars were to be spaced accurately; and, finally, that the contractor was to clean all stains and concrete droppings from the metal work.

The motorway stringers at the side of the roadway were used as a runway for the concrete mixing equipment. A standard-gauge track was laid on these stringers for some distance, to accommodate the contractor's plant, which he placed on several railway flat cars. This plant consisted of two large elevated hoppers, connected by a cable carrying a conveyor bucket, and a belt conveyor for handling finished material, and similar appliances. Much time was consumed in the arrangement of this equipment which was soon found to be unsuitable. The company's engineers then designed a new and simple plant, the

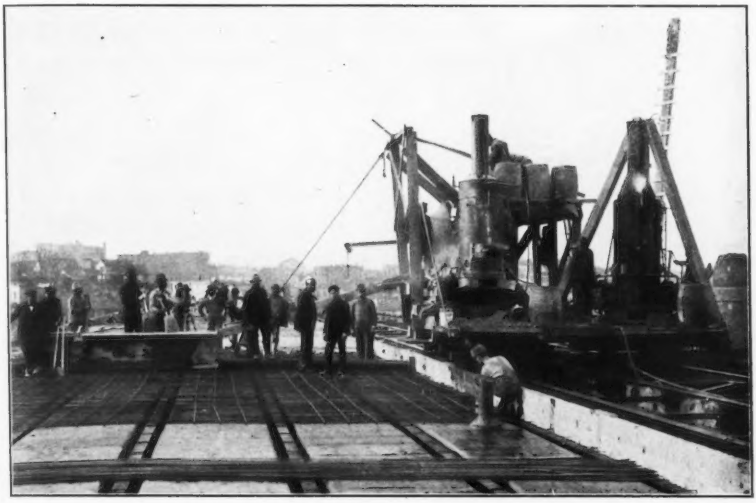


FIG. 1.—ARRANGEMENT OF CONCRETE PLANT FOR LAYING FLOOR

FIG. 2.—FINISHED FLOOR OF MULBERRY AND HICKORY STREET APPROACHES.

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old elaborate one was dismantled, and five of the smaller plants were quickly built and put into successful operation.

The plant is shown in Fig. 1, Plate VIII; it consisted merely of a platform of timbers, mounted on wheels to run on the motorway stringers, supporting a mixer, a derrick, and an engine. The derrick raised the aggregates from below with a skip box, and a metal-lined chute extending over the roadway received the concrete from the mixer. The total cost of one plant, including all fittings except the mixer and the engine, was $700.

Forms.—Several arrangements of forms were tried, and, as these made up a considerable part of the floor concrete cost, they merit special attention.

The contractor first furnished all-metal forms, made of corrugated sheets pressed to shape and covered with light sheet iron. They were made in sections 5 ft. wide and about 5 ft. long, the longitudinal junction of various sections being immediately under the stringers. Along each of these lower edges there was fastened a small angle having holes in its vertical leg. Keys were provided to pass through these slots to fasten adjoining forms together laterally. Provision was made for the forms to lap about 1 in. longitudinally, but there was no way of fastening them together. To support these forms, three joists were placed from cross-beam to cross-beam, and three sections of form were laid thereon. Numerous imperfections were soon evident. The form had almost no lateral stiffness, each unit having an individual shape. At best, the surface presented was not true; the correct distance was not left on each side of and below the I-beams; there were openings and cracks at the transverse joints, as these could not be made to lap as intended; many pieces had to be held during adjustment; and, also, the forms were easily displaced by the ordinary walking about and working on them.

Owing to the non-delivery of the metal forms, the contractor had already begun to build wooden forms of much the same type, that is, of sections to meet under the stringers and of such length that three were put between adjacent cross-girders. They were built of 1 by 4-in. frames spaced 18 in. from center to center, sheeted with ½-in. boards, and covered with No. 14 sheet iron. These forms were hung on the stringers with two ¼-in. round, soft iron rods on each side. After adjusting the form to position, the hangers were bent around the upper

flanges of the I-beams. To remove the forms, the hanger rods were cut with heavy clippers close to the sheeting. These wooden forms were light and stiff, but they, too, showed objectionable defects. There were no means of really fastening the various units together, and there were presented, not only the transverse cracks of the metal forms but also a longitudinal crack under each stringer, and these latter were almost inaccessible for inspection or adjustment. Nothing was found to prevent water and cement from leaking out.

The company's engineers then designed a form, and the use of the others was discontinued as soon as they could be replaced. The finally acceptable form consisted of two parts: U-shaped troughs which were placed under the stringers; and covers which extended from trough to trough between stringers. They were of wood, 1 by 4-in. frames, with ½-in. or ⅜-in. sheeting, and covered with No. 16 or No. 18 sheet iron. The troughs were 13 ft. 10 in. long, or just enough to go easily between the cross-beams; the covers were of various convenient lengths, from 4 to 6 ft.; and the outside curb faces were 18 ft. long. To support the troughs, ¾-in. round iron hangers were used, a loop-shaped piece being put around the 2 by 4-in. sill on the bottom, and each leg being bent over the upper flange of the I-beam. Four or five ties were used for each section. To hold this trough in correct position and prevent rotation, briquette spacers were used. These were made to fit the lower curve of the form, and had a shoulder on each side at the top to fit the lower flange of the I-beam.

A nest of wooden moulds was provided, and briquettes of sand and cement were made in advance of the need, so as to be thoroughly hardened. One briquette was placed at each end of each trough (thus closing it against leakage) and one under each hanger. When the hangers had been drawn reasonably tight and bent over, the trough was so rigid that a man could walk along one edge.

The cover forms, it was found, could be handled with a lap at one end, and, when the troughs were in position, could be rapidly laid, producing a continuous unbroken surface. The small remaining vertical hole (8 by 12 in.) at each floor-beam was covered with a piece of No. 8 sheet iron. The forms for the outside faces of the curbs were made of ⅞-in. plank. These were stood on the already erected curb trough, adjusted to position, and nailed lightly. A horizontal hanger held them laterally, and they could be made truly vertical.

These forms proved to be highly satisfactory, presenting smooth, rigid, correctly-shaped surfaces, with very few chances of leakage. They were easily altered to meet conditions, and stood rough treatment well. Their erection and removal was simple, any part being handled by two men without extra scaffolding. Fig. 2, Plate VII.

The briquette detail was one of the most advantageous features, and was developed by a prior use of wooden blocks, which gave trouble by breaking out the concrete as they were removed with the form. The cement briquettes remained tightly in place, and panel after panel of floor was built without a place to touch up below the beams.

The cost of building 600 lin. ft. of these forms for the 30-ft. roadway was:

Lumber	$1 160
Sheet iron	650
Nails and sundries	165
Labor	1 300
Contractor's profit	325
	$3 600

The first cost was $6 per lin. ft., or $0.20 per sq. ft. between curbs. Forms used five times cost 4 cents per sq. ft., or $1.30 per cu. yd. Many of these forms were used oftener than five times. The fact that they were built in great haste doubtless increased their cost somewhat.

Making and Placing Concrete.—Owing to difficulty in securing a sufficient quantity of 1-in. stone from local crushers, the use of flint screenings from zinc mines was permitted. Although this material is clean, hard, and sharp, and is somewhat graded in size, needing but little sand, there are objections to it. A few moments after it had been deposited quite wet, the water appeared to drain away, leaving the upper stones clean and dry, without any mortar coating whatsoever, and the entire mass appeared to be inert and "dead". This appearance was evident even while the concrete was still green enough to be dug through with a stick. After several days the main body became hard and compact, but on top there was perhaps ¼ in. of loose stones. Various mixtures were tried, to see if the concrete could be made to remain "quaky" for a time, but it could not be done. Yet experiments on the

tensile strength gave creditable results, and investigations by cutting through slabs thoroughly set showed the concrete to be very hard. About half the floor concrete was made of this flint, and the remainder of crushed limestone. The latter, of course, could be flushed with mortar, and gave normal results.

All concrete materials were delivered on the ground under the viaduct. Skip boxes were filled with a batch of correctly proportioned aggregates and raised to the mixer above. The concrete was dumped into a trough and slid down to be distributed over the floor. To expedite the filling of the small space below the I-beams, cement mortar was used to a height of about 2 in. above the lower flanges.

The reinforcing bars were spaced and held in position as follows: Round ¾-in. iron rods were laid longitudinally on the forms, about 5 ft. apart, and on these the lower transverse bars were placed. The lower longitudinal bars were then laid in position and fastened at numerous intersections with wire ties. When some 3 in. of concrete had been spread, a rope was fastened to the ends of the ¾-in. rods and a strong pull slipped them out longitudinally from the finished work. A very minute inspection of the whole floor failed to show a single instance of a lower bar exposed.

Combined supports for the upper bars and guides for finishing the upper surface consisted of 2-in. joists having small angles fastened on each side and a series of hooks in the lower edge. They were placed longitudinally with the structure and 10 ft. apart, and were held on small blocks resting on the stringers. The hooks supported the transverse upper bars in correct position. A gauge, sliding along the angles, gave the desired surface shape to the concrete.

The concrete was first tamped with a large flat tool and, after hardening somewhat, was retamped with a corrugated tool. The latter was made of wood with parallel cleats, ¾ in. wide, 1½ in. deep, spaced 2 in. apart. The cleats were sharpened slightly. With proper manipulation this produced light corrugations, in varying directions, of $\frac{3}{16}$ to ¼ in. variation, thus giving a satisfactory bond for the asphaltum. Scattering loose stone and tamping it in was tried, but the former roughening was superior. Neither method gave entire satisfaction where flint was used.

The plants were moved forward as each section of about 15 ft. of roadway was completed. On an average, the five plants placed from

300 to 350 lin. ft. per day; the best day's run for one gang was 98 lin. ft., or 90 cu. yd.

The floor was finished level to the edges, and the curbs were built later, in the usual way. Bars projected upward into this curb every 12 in. and bound it to the body of the work.

With all types of forms, and to a pronounced degree with the first designs, there was considerable leakage from the wet concrete. This dripping water carried such a quantity of cement as to leave a whitish deposit on the metal where it drained, and in certain cases spattered down on the ground under the structure. Probably the actual loss of cement was small, but although it was easy to know how much cement went into each yard, or each day's run, no scheme was discovered to ascertain how much actually remained in the concrete. This cement was not cleaned from the metal promptly, and the cleansing later proved troublesome. The plan was to wash the metal with a strong stream of water immediately after the concrete was placed and until it ceased to drip. Adequate pumps and lines were not installed to do this, and finally much cleaning had to be done with wire brushes and scrapers, necessitating some repainting.

The entire concrete floor was built in 67 working days, during which there was some freezing weather. The contract provided for heating any or all concrete materials, but none was heated other than some sand and this merely to thaw frozen lumps. However, wagon sheets were spread over the green concrete each night, and, when hard weather was expected, these were covered with stable sweepings. No bad effects were found on the floor proper, but some portions of curbing were frozen and had to be torn out and replaced. Some of the concrete was laid when the temperature was 31°, and the following days were about the same, yet in 10 days' time asphalt rollers were run over the work.

The 30-ft. roadway contains 24.6 cu. ft. per lin. ft. and the 38-ft. roadway 33 cu. ft. per lin. ft., or an average for both of 0.84 cu. ft. per sq. ft. of area between curbs. The average amount of reinforcing is 2.67 lb. per sq. ft.

The main viaduct contains about 8 000 cu. yd. of concrete. The contract cost, exclusive of cement and bars, was $6.085 per cu. yd. The cost of completed floor, not including paving, was about 36½ cents per sq. ft.

Paving.—The paving is of Bermudez asphalt. The following results are from average analyses:

Specific gravity	1.098	
Insolubles	2.0%	
Soluble in 88° naphtha	63.0	
Soluble in 62° naphtha	85.0	
Penetration at 77° Fahr.	38°	(Dow.)
Fixed carbon on ignition	14.5%	
Ductility	20.	(C. M.)

This hard asphalt was softened by the addition of asphaltic petroleum residuum which tested as follows:

Specific gravity, Baumé	11.25
Boils	83° cent.
Flashes	173.5° cent.

From 18 to 20 lb. of this oil were mixed with 100 lb. of asphalt. The hardness of the resulting mixture at 77° Fahr. varied from 67 to 78° (Dow).

It is rather difficult to secure just the sand desired for asphalt in Kansas City, as the Missouri River sand is somewhat water-worn and the finer Kaw sands contain some loam. Pulverized limestone and cement were added to the sand to give the required 200-mesh material.

The following mix was taken as a criterion, although the specifications gave somewhat more latitude:

Bitumen			9.5%	to	11.5%
Sand passing		200	6 %	"	16 %
"	"	100	8 %	"	16 %
"	"	80	10 %	"	16 %
"	"	50	14 %	"	34 %
"		40	9 %	"	16 %
"	"	30	6 %	"	10 %
"	"	20	3 %	"	7 %
"	"	10	2 %	"	4 %

The paving material was given close inspection, both at the plant and on the work. Samples, taken daily, were analyzed and sifted, and daily directions were given to the mill. On the viaduct, samples

were taken from 1 in every 5 to 15 loads, depending on the rapidity of delivery, etc.

The rolling was done carefully with 6 and 8-ton rollers. Where necessary, the surface was ironed, with especial attention to securing adequate drainage. The entire pavement was placed in 20 days.

The iron trough expansion joints were filled with the ordinary mixture. At some of these the wearing surface cracked during cold weather, and in the later work thin separators were inserted to make these openings straight across the roadway. These joints are acting as expected, and are considered very satisfactory.

Asphalt was not laid on the new embankments, but a Macadam pavement on a very heavy base was substituted and finished with oil.

The contract cost for the asphaltum was $1.05 per sq. yd., and for the Macadam, $1 per sq. yd. Thus the concrete floor with pavement cost about 48 cents per sq. ft.

Street-Car Deck.—The ties of the street-car deck are of creosoted, short-leaf, Southern pine, 6 by 8 in., laid flat, and spaced 13 in. from center to center. Every fourth tie extends across both tracks, thus serving to brace the whole deck together and support a possible runway. Guard timbers are dapped 1½ in. on the ties and are set 11 in. beyond the gauge. On curves, the ties are on edge, the variation in the depth of the daps giving the outer-rail elevation.

All the timber was surfaced on four sides, and finished to full size. The general creosoting specifications were the same as already given for piles, from 10 to 13 lb. of pure dead oil of coal-tar being used per cubic foot of timber. The dapping, boring, and cutting were done before creosoting. The deck timber cost delivered $45 per 1 000 ft., B. M.

A small derrick car, equipped with a gasoline hoist running on the stringers, raised the material to the decks from its distributed position below. The ties were set to approximate place and the guard rails laid. Wooden rams were used to drive the guard rails down on the ties, to shift the ties to fit the guard-rail daps, and to settle them on the stringers. Then the hook-bolts and guard-rail bolts were inserted, and the deck was tightened to place with socket wrenches on auger handles.

The rails are of 70-lb., Am. Soc. C. E. section, 30 and 33 ft. long, laid with broken joints, fastened to every tie with two ordinary 5½-in. spikes. Six-hole angle-bars were used.

The tracks are bonded by two copper bonds at each joint, and set up with a screw compressor. For each bond, two $\frac{7}{8}$-in. holes were drilled through the rail web. Some of these holes were drilled with hand-ratchets, but as the rails were very hard and the drilling was slow, a machine drill was substituted. This consisted of a $1\frac{1}{2}$-h.p. gasoline engine, set on a push-car, operating a flexible shaft, which was attached to an automatic-feed drill working in a yoke that fitted over the rail. This machine cost only $100, in addition to the engine, and gave creditable results. It would drill the four holes at a joint and move forward in 12 min. A very weak solution of sal soda was used for lubrication.

The cost of laying deck timber was about $5 per 1 000 ft., B. M.; for laying rails about 4 cents per lin. ft. of single track; the work of bonding cost about 50 cents per joint.

The usual overhead trolley is used. The poles are of iron, from $6\frac{1}{2}$ to 5 in. in diameter and 20 ft. high above the ties. They are placed between the two tracks, about 90 ft. apart, and support cross-brackets for the wires.

Hand-Rail.—The hand-rail was designed for strength, simplicity, and architectural harmony with the structure. It consists of square, cast-iron posts, 9 ft. apart, supporting horizontal rails of ordinary pipe, the diameter of the upper one being 3 in. and the other four $2\frac{1}{2}$ in. Each post is fastened by two bolts embedded in the concrete floor, the upper of which has two nuts, in order to provide for the adjustment of the post. The pipe rails pass through holes in the posts, fitting rather loosely, and are joined by standard unions and made continuous. At each post, and for each pipe, two small steel wedges are driven into seats, gripping and pinching the pipe and holding it tightly in place.

Thus the hand-rail could not be erected until the floor was finished, and then the posts and rails had to be put up simultaneously. A small two-wheeled truck was used to set up the posts. The forward end of this truck had a cantilevered fork on which a post was supported as it was drawn on to the projecting bolts. The railing pipes were inserted consecutively and jointed together as the successive posts were erected.

When several hundred feet of railing was in place the posts were adjusted to points set by a transit, thus bringing the upper rail to a true line. The cost of the hand-rail is included in the general schedule price for superstructure metal erected.

Lighting System.—The annoyances of a trip through the bottoms are aggravated by darkness, and an adequate lighting system for the accommodation of night travel was considered highly desirable for the viaduct. Natural gas is available, and was considered, as also were other plans for lighting, but an incandescent electric system was selected. The results are exceedingly satisfactory; the illumination is uniform and not glaring, yet so bright that a newspaper can be read anywhere on the roadway.

Every fourth hand-rail post is extended above those intermediate, and supports a 16-c.p. Tantalum lamp, set in a vertical, moulded, mica socket, surrounded by a glass globe 10 in. in diameter. There is thus a light every 36 ft. along each hand-rail. The lights are uniformly 8 ft. 6 in. above the pavement, and are staggered on the two sides, making one lamp for each 18 lin. ft. of roadway.

The wiring is of the three-wire, 250 to 500-volt system, with two lamps in series, connected in three circuits so that any third of the lamps may be used without changing the factors of symmetry and distribution. Certain main posts have lamps wired on the three circuits so as to show light regardless of which third of the system is in use.

In a central office is installed a switch-board, a motor generator, and certain other machinery. From this office are run two sets of three circuits, each with a common neutral for each set, to furnish current for the lamps of half of the structure, and two similar sets of three circuits each to furnish current for the lamps of the other half. The two outside wires of each circuit, together with its series wires, are run in one metal conduit, and the neutral for each set of three circuits is run on glass insulators.

The conduits and wires are supported on short wooden cross-arms fastened on the top of each floor-beam between the two outermost stringers on each side. They are thus well sheltered, on each side and above, by the concrete floor.

From the office to the structure the four neutrals are combined into two wires, each of solid copper, No. 10, B. & S. gauge. Elsewhere, each neutral consists of a solid copper wire, No. 12, B. & S. gauge. The wires connecting two lamps in series are of the same character, and of No. 14 gauge. Where placed on glass insulators, the neutral wire is of single-braid, Okonite insulation, and all feeders and branches run in metal conduit have double-braid insulation. Joints in the wires

are soldered and double-taped with Okonite and friction tapes. The conduits are of loricated metal, and where branches are taken off, Crouse-Hinds condulets are inserted.

The feeder wires of the lamp are led down through the post to a fuse-box fastened near its bottom, in which is a 6-ampere cartridge fuse. To reach the fuse-boxes from below the floor, a 2-in. pipe was placed in the concrete at each lamp-post, and through this is passed a light $\frac{3}{4}$-in. metal conduit, bent to prevent the entrance of water, and bushed at each end.

All branch wires running under feeders from the conduit are encased in continuous lengths of Alphaduct. Corresponding with the variation of the size of the wires, the conduits are varied, the six carrying main feeders being $1\frac{1}{4}$, 1, and $\frac{3}{4}$ in. in diameter, respectively.

There was serious trouble in entering the several wires into the conduits, because of their comparatively small size; slightly larger conduits would have simplified the task.

The company furnished the lamps, globes, and certain castings, but the contractor installed them, and furnished and installed all other equipment for $28 000. There are about 500 lights on the main viaduct.

6.—Approaches and Laterals.

As has been stated, legal complications prevented the main lateral on Mulberry Street from being built as at first planned, and it became impossible to effect a landing at 12th and Liberty Streets. This lateral was always considered of vital importance to the success of the enterprise as a whole, and when the original plans had to be modified it was decided to build to 12th Street—two blocks east of the former corner—and there install electric elevators of sufficient size and capacity to raise to the deck of the structure any vehicle with its horses. Further delays of a legal nature arose, however, and at present, December 1st, 1908, the Mulberry Street lateral is built to the corner of Mulberry and 9th Streets, and two elevators are installed there. The Hickory Street branch of this lateral divides from the main branch about 300 ft. from the main viaduct, runs down on a 3% grade, turns under the former, crosses Mulberry Street, and ends at 9th and Hickory Streets. This branch is shown on Fig. 2, Plate VIII.

These laterals provide only for the highway loading, the Mulberry Street portion having a roadway 24 ft. wide, and the Hickory Street

branch a roadway 20 ft. wide. The pavement floor, hand-rail, and details are the same as for the main viaduct. The detailing of the steelwork is very similar, but of very much lighter construction; it is a two-column structure with longitudinal girders, transverse beams, and longitudinal stringers. No towers or longitudinal bracing are used; the columns are fastened down with four anchor-bolts which provide the necessary stiffness in every direction.

The foundations are principally of small concrete pedestals resting on sand. They are placed at a sufficient depth below the street grade to allow for cellar construction. Two of the elevators have been installed. Each has a counter-weighted moving platform, 11 by 32 ft., lifting a total distance of 40 ft. 9 in. They are designed to carry 22 000 lb. at 50 ft. per min., or 11 000 lb. at 100 ft. per min. One 50-h.p. motor is used for each elevator. Automatic gates are provided above and below and on the platforms.

These two approaches were finished and opened to traffic a few months after the completion of the main structure.

The distances saved by the viaduct are shown by the following computation:

Distance between Eighth and Wyandotte Streets and Fifth Street and Minnesota Avenue.

By Metropolitan Street Railway line.....	11 950 ft.
By Sixth Street Viaduct.....	8 400 "
Saving.....	3 550 ft.

Distance between Eighth and Wyandotte Streets and Fifth Street and Minnesota Avenue.

By Metropolitan Street Railway line.....	16 000 ft.
By Sixth Street Viaduct.....	11 850 "
Saving	4 150 ft.

7.—Conclusion.

The entire construction of the main viaduct was completed in 18 months; in fact, the work was done with such dispatch that several contracts were finished ahead of their allotted times. The substructure was ready for steel in 7 months. The steel erection occupied 10

months, the concrete floor was in construction 3 months, and the asphalt was laid in 1 month.

It is not possible, without lengthy explanatory detail, to give exact distributed costs of the complete structure. These figures have been compiled and compared minutely with the preliminary estimates. When due allowance is made for various changes, additions, and deductions, which originated after the first estimates were prepared, a very close agreement is seen. The greatest variation of actual cost to estimated cost is to be found in the items "right of way" and "legal expenses", both of which are materially in excess of the estimates.

Contracts were made on the basis of unit-price payments for quantities, and provision was made for all anticipated contingencies. The extras on all contracts were performed at actual cost plus 10 per cent. The total amount of such bills on the two principal contracts was about $\frac{8}{10}$ of 1% of the total final estimates.

The total amounts paid to the contractors, and the total cost to the company, of the principal physical features of the structure are given below, with the unit-prices for the work.

The costs refer to the Main Viaduct, which includes the Kaw River Bridge.

Substructure:		
All pedestals, abutments, earth work, etc.	$283 000	
Three piers in the Kaw River	98 000	
		$381 000
Superstructure:		
Steel erected and painted	$785 000	
Concrete floor	95 000	
Pavement	35 000	
Street-car deck and trolley	86 000	
Lighting system	28 000	
		1 029 000
Right of way		227 000
Total		$1 637 000

Unit-Cost of Work.—

Concrete in pedestals, including excavation, forms, etc. .....$10 per cu. yd.
Concrete in shafts of river piers..... 11 " " "

Mass of foundations of pneumatic piers. (This comprises the gross volume of base, including timber, iron, concrete, and cost of sinking) .....$18 per cu. yd.
Anchorage metal in place .....4¾ cents per lb.
Creosoted piles furnished at site .....46.40 cents per lin. ft.
Driving creosoted piles, measured below cut-off .....30.25 to 31.35 cents per lin. ft.
Concrete piles in place.....88 cents per lin. ft.
Structural metal delivered, erected and painted .....3.31 cents per lb.
Construction of concrete floor (cement and bars furnished)..... $6.085 per cu. yd.
Asphalt pavement..... 1.05 per sq. yd.
Laying ties for street-car deck (all material furnished)..... 5.00 per 1 000 ft., B. M.
Laying, bolting, and spiking rails.....4 cents per lin. ft. of single track.

Costs per Linear Foot.—

Viaduct with 38-ft. Roadway, and Two Car Tracks.

Substructure	$38.15	per lin. ft.
Steel	92.60	" "
Floor and pavement	18.95	" "
Street-car deck.....	10.75	" "
Lighting system.....	3.50	" "
Total cost.....	$163.95	per lin. ft.

Viaduct with 30-ft. Roadway, and Two Car Tracks.

Substructure	$38.15	per lin. ft.
Steel	88.75	" "
Floor and pavement.....	15.35	" "
Street-car deck.....	10.75	" "
Lighting system.....	3.50	" "
Total cost.....	$156.50	per lin. ft.

Kaw River Bridge, 30-ft. Roadway and Two Car Tracks.

Substructure	$163.00	per lin. ft.
Steel	198.50	" "
Floor and pavement	15.35	" "
Street-car deck	10.75	" "
Lighting system	3.50	" "
Total cost	$391.10	per lin. ft.

The small difference in cost between the 30-ft. and the 38-ft. roadways is accounted for by the fact that the former is built strong enough in all details to allow for widening it to 38 ft. by lengthening the roadway cantilevers.

Considering both the roadway and the total width of both street-car tracks as upper surface area, the costs per square foot of this area were:

For 38-ft. roadway portion	$2.66	per sq. ft.
For 30-ft. roadway portion	2.93	" "
Average for viaduct proper	2.835	" "
For Kaw River Bridge	7.32	" "

Personnel.—This viaduct has been a work of many minds and many hands, and any list of those associated with it must necessarily be incomplete. The structure was designed by, and constructed under the supervision of, the firm of Waddell and Hedrick, composed of J. A. L. Waddell, and Ira G. Hedrick, Members, Am. Soc. C. E., Consulting Engineers. (This firm has since dissolved.)

Mr. J. H. Thompson, of New York, was Chief Engineer, and had, as his local representative, Mr. E. S. Whitney. V. H. Cochrane, Assoc. M. Am. Soc. C. E., was Engineer in Charge of Shop Inspection. The writer served as Resident Engineer for the Consulting Engineers, and was in general charge of the construction.

The general contract for the substructure was in the hands of Mr. James F. Halpin of Kansas City, under whom were various subcontractors. The pneumatic piers were built by Kahmann and McMurray, of Kansas City. The concrete piles and most of the timber piles were driven by The Foundation Company, of New York, under the personal direction of the late George Adgate, M. Am. Soc. C. E. The steel was furnished and erected by the Riter-Conley Manufactur-

ing Company, of Pittsburg, with Mr. John P. Wagner as local representative.

The concrete floor was built by The Expanded Metal and Corrugated Bar Company, of St. Louis, under George R. Heckle, Assoc. M. Am. Soc. C. E. The asphalt paving was laid by the Parker Washington Paving Company, of Kansas City. The laying of the street-car deck, certain substructure, and other various work was done by H. C. Lindsly and Son, of Kansas City. The lighting system was designed by Messrs. Weeks and Kendall, Consulting Engineers, and installed by The Squire Electric Company.

The writer remembers with appreciation the efforts of a number of assistant engineers and inspectors, who labored zealously and efficiently. Grateful thanks are due to many gentlemen for aid in collecting the foregoing data, particularly to Messrs. Waddell, Hedrick, and Cochrane. L. R. Ash, Assoc. M. Am. Soc. C. E., was identified throughout with the designing and preparation of the plans.

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AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

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SAMPITTIC SURFACING.*

By Walter Wilson Crosby, M. Am. Soc. C. E.

To be Presented April 21st, 1909.

For several years the writer has been studying the uses (and results) of hydrocarbon compounds in road making, and has been impressed with the lack of standards by which to judge the various so-called "bituminous cements" offered for sale, as well as with the pressing need for such standardization. With the increased use of this cement has come the inevitable increase in the variety of trade compounds; moreover, with the change in traffic conditions, highway engineers tend to conclude that the need for such a proper cement, and its future use, are greatly increased.

The writer, from personal inspection, is familiar with most instances of the use of hydrocarbon cements in road building in the Eastern States, and thinks it will not be disputed that cements of considerable variety have been used and many differing results have been secured.

Under different conditions, occasionally, similar results have occurred; and, under similar conditions, varying effects have been obtained. Analyses of the causes for these variations seem to indicate that differences in the hydrocarbon compounds used are the main

* Gr., psammos, sand; pitta, pitch.

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sources of variation, and that, if proper standardization of the cement material could be secured, one might hope for uniformity of results. To this end the writer, for more than a year, has been conducting a search for information concerning the composition, from the standpoint of the highway engineer, of the hydrocarbon compounds available, and the effects, when used on the road, of certain constituents known to exist. He regrets to report a woeful lack of published information on the subject, this, apparently, being due to the complexity of the compounds dealt with, and the unscientific way in which many who are most familiar with the use of the material in question have made and recorded their experiments. The results attained by the writer are far from perfect, on account of the limited time at his disposal for this work, and, quite likely, his limited abilities in this direction. He hopes, however, that by directing attention anew to the subject, and possibly by furnishing a concrete starting point for further work, his efforts will not have been in vain.

It will be recalled that, not so many years ago, it was the custom to specify cement of a particular brand, or cement which, in the opinion of the engineer, was equal to the product of a certain named firm. To-day lime cement is a standardized article, and such specifications would be looked upon with contempt.

Is it more than fair to hope that the standardization of bituminous cements may follow shortly; and to believe that, not only is it possible that within a few years the engineering profession will be relieved of criticism for such specifications as "Borrowin's Pilgrim Brand Joint Filler," "Rotto Asphalt," "Stickulithic Processes," or "equally good" thereto?

The specifications for "Sampittic Surfacing," offered herewith, may be used as a starting point for discussion, principally because it seems to the writer that the simplest form of the use of a hydrocarbon cement is thus presented.

Further, it seems possible that, if a proper standardizing of a base is secured, modifications or solutions of this base, required under different conditions and for different purposes, may easily be made by simple changes in the inserted figures, as the necessity may dictate.

The writer will gladly welcome discussion of the foregoing, and of the specifications appended, especially those clauses relating to the "Pitch Compound."

SPECIFICATIONS.

Sampittic Surfacing.

281.—Sampittic surfacing shall be used wherever provided for in the plans or directed by the Engineer. The width and thickness required at different points shall be those shown on the plans, or designated by the Engineer.

282.—This sampittic surfacing will usually consist of one 6-in. course of a thorough mixture of pitch and sandy soil, to be formed on the roadbed after the latter has been properly graded and shaped.

Roadbed Material.

283.—The roadbed for the sampittic surfacing shall consist of the natural sandy roadbed prepared and shaped as shown on the plans. Where the natural roadbed is found, during the work, to consist of material which, in the opinion of the Engineer, is unfit for this form of construction, the Contractor shall remove as much of this unfit material as may be deemed necessary by the Engineer and shall replace it with sandy material which, in the opinion of the Engineer, is suitable for the purpose. All unfit material thus removed will be measured and paid for as "Excavation."

Cuts and Fills.

284.—In cuts and fills, unless otherwise specially directed, the roadbed shall be graded to a width of —— feet, and shall be free from spongy and vegetable matter, roots, and stumps. The portion of the roadbed prepared for the pitch surfacing shall be —— feet wide, and shall be brought to the grades and cross-sections shown on the plans. The fills shall be —— feet wide on top.

Old Earth Roadbed.

285.—Where no change from the present grade of those portions of the road, not already surfaced, is shown on the profile, the roadbed shall be shaped to the proper cross-section; and slight elevations, with contiguous depressions, shall be removed, so as to form an even and smooth surface.

Shape.

286.—The shape of the finished road shall be that shown on the accompanying plans, and shall have a cross-slope of —— inches to one foot.

Pitch Compound.

287.—The pitch compound to be used shall be a hydrocarbon, refined from tar or petroleum, or made from natural asphaltic pitches, by the use of a proper solvent or flux.

It shall be uniform in character, and shall possess the following characteristics:

a. The percentage of free carbon shall not exceed ten.

b. The percentage of fixed carbon shall not exceed twenty.

c. The percentage of paraffin shall not exceed two.

d. When 20 grammes of the material are heated on a flat-bottomed dish, 2¼ in. in diameter, for 24 hours, in an oven, the interior of which is maintained at a uniform and constant temperature of 105° cent., the loss shall not be more than 15 per cent.

e. A sample of the pitch compound proposed to be furnished will be evaporated until it has been hardened to show a penetration, by the Dow machine, of 50 cm. at 25° cent. This pitch must show a ductility, by the Dow machine, greater than 50 cm. at 25° cent., and greater than 10 cm. at 10° cent.

f. When an amount equal to one-half the percentage of loss, under the requirements of Clause *d*, shall have been distilled from the normal pitch compound, the residue shall show a consistency of not more than 40 cm. at 35° cent.

g. The consistency of the pitch compound for this work shall be such that 50 cu. cm., at a temperature of 100° cent., will flow through an Engler instrument in not more than 75 sec.

h. When the pitch compound proposed to be used shall have been evaporated to a consistency which will show a penetration of 50 cm. at 25° cent., the percentage of this residue to the whole sample of normal pitch compound shall be not less than 60.

i. The melting point of this residue shall be not less than 65° nor more than 85° cent.

j. The pitch compound proposed to be used shall contain no constituents which can be removed by distilling it up to 170° cent., nor more than 20% by distillation between 170° and 270° cent.

k. All pitch compounds must be free from dirt or adventitious matter in excess of 2% of the compound in its normal state.

l. Pitch compounds which, in the opinion of the Engineer, have

offensive characteristics, by reason of their odor or otherwise, will not be accepted.

m. Each lot of pitch compound furnished under this contract will be subject to the tests mentioned, and shall show by these tests the characteristics specified. The tests will be made on samples taken from each lot as it arrives, and the results of the tests on the samples will be considered as characteristic of the lots from which the samples were taken. The rejection of a sample for any reason will cause the rejection of the lot from which the sample was taken. The samples will be taken, by the gallon, from tank cars, by the Engineer or his authorized assistant. If the pitch compound is delivered in barrels, a pint will be taken from each of eight barrels selected at random by the Engineer from each carload of the shipment, and these pints will be thoroughly mixed together to form the sample to be tested.

288.—All pitch compounds, as well as all other materials, used under this contract, shall be subject to the approval of the Engineer, and pitch or other material disapproved by him shall not be used. Any disapproved material shall be removed promptly and replaced by approved material, by and at the expense of the Contractor, on notice to him from the Engineer.

Spreading and Mixing.

289.—When the roadbed, after being prepared and shaped as herein described, does not consist of material in a finely divided state to a depth of at least 6 in., if deemed necessary by the Engineer, it shall be put in such condition by thoroughly plowing, harrowing, and cultivating to a depth of at least 6 in.

290.—On the roadbed, prepared as herein described, the pitch compound shall be spread uniformly by using a sprinkling cart of a pattern approved by the Engineer, or by such other means as may be so approved, and in a quantity not less than 1 gal. nor more than 1½ gal. per sq. yd. of surfacing. Unless otherwise approved in writing by the Engineer, the pitch compound shall be heated to a temperature of not more than 150° cent., and applied at a temperature of not less than 80° cent.

291.—Immediately after an application of pitch compound has been made to a section of road, as above described, the pitched material shall be thoroughly turned over with plows or harrows and mixed so

that none of the unmixed pitch is visible on the surface. A second application of pitch compound will then be made as before. The pitched material will then be mixed thoroughly by the use of plows, harrows, road scrapers, and such other means as may be necessary to insure a complete, uniform, and thorough mixing of the pitch compound and the sandy material of the roadbed. To secure this result, additional pitch compound shall be supplied, where necessary, by the Contractor, in the manner and under the conditions above described. Uniformity of mixing is most important, and must be secured, and pockets of unusually sandy or excessively pitchy material will not be permitted. When finally the roadbed material is mixed satisfactorily, in the opinion of the Engineer, the surface shall be carefully shaped, by raking or otherwise, to the cross-section, grades, etc., shown on the plans, due allowance being made for settlement.

Rolling.

292.—The pitched material shall then be rolled by a "tamping roller" of a pattern approved by the Engineer. This roller shall have toothed concentric rings, or shall be a drum studded with spikes, or shall be of a design which will insure the thorough compaction of the pitched material from the bottom up. The rolling will be continued until the utmost degree of compaction is secured, and until the spikes or teeth of the wheels do not penetrate the surface more than 2 in. During the rolling, and if deemed necessary by the Engineer, the road shall be sprinkled with sufficient water to insure the thorough compaction of the surfacing.

Finishing Coat.

293.—When, in the opinion of the Engineer, the pitched material shall have been sufficiently compacted, as above described, a third coat of pitch compound shall be applied, as hereinbefore described, and in a quantity not less than ½ gal., nor more than 1 gal., per sq. yd. The surface shall then be lightly harrowed or raked, or both, to insure a thorough mixing of the pitch compound with the loose material remaining in the road, and shall be raked and trimmed to the cross-section and grades desired. The road shall then be thoroughly rolled, until firm and hard, with an ordinary steam roller, weighing not less than 5 tons. Any unevenness or depression shall be remedied by the addi-

tion of properly mixed pitch compound and sandy material of a character similar to that in the roadbed, as may be deemed necessary by the Engineer. Should spots in the road show an excess of pitch under this last rolling, sufficient sandy material to absorb the pitch shall be dusted over them carefully and evenly.

Price.

294.—The price agreed to be paid for pitch surfacing shall include all labor and material necessary to do the work herein specified under pitch surfacing, and shall also include the repairs necessary, in the opinion of the Engineer, for a period of two months from the day of the acceptance of the work, which repairs shall be made with the same materials and in the same manner as the construction above specified.

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THE MAXIMUM WEIGHTS OF SLOW FREIGHT TRAINS.

By C. S. Bissell, M. Am. Soc. C. E.

To be Presented April 21st, 1909.

It is hardly necessary to mention the great and growing importance of the subject of train weights. Study of the problem has given rise to many different forms of investigation which are constantly appearing in the railroad journals and which are indicative of greater refinement in the making up of trains in the future. Most writers have endeavored to treat the subject in its entirety, comprehending the operation of both slow and fast trains, whereas the object of this paper is to present only the case of slow freight trains, and to outline briefly the history of the various steps which led to the conclusions finally reached.

The writer was called upon to formulate some method of estimating operating expenses, over proposed revisions or projected lines not constructed, such as would permit of the intelligent comparison of expenses on two or more projects or routes of haul. The locomotive was of a type weighing, with its tender, 168 tons, of which 173 000 lb. rested on the driving wheels; and the train was to consist of cars weighing 20 tons each when empty, with a capacity of 50 tons. The

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speed was not to exceed 10 miles per hour on ruling grades, and no benefit derived from momentum was to be considered in fixing the train weight. The resistance due to curvature was to be allowed for by compensating the ruling grades at 0.05% per degree of curve.

The first principle of train resistance, namely, that a given train weight confined in a few cars requires less tractive power to move it than the same weight composed of many empty cars, led the writer to a form of equation involving the relation between the dead weight and the lading of the cars, which is evidently the same as if the average car weight or the actual number of cars had been used.

The tractive power of the locomotive was determined as the average amount which would usually be developed within the adhesive power of the driving wheels, considering that atmospheric conditions and steaming qualities remained practically constant at normal values. The tractive power was to be reduced by the proper amount for a constant speed of 10 miles per hour.

With these ideas, which indicated the general form of the equation, the paper* on "Virtual Grades for Freight Trains" by A. C. Dennis, M. Am. Soc. C. E., was consulted, with the result that the train resistance was taken at 4 lb. per ton of train weight for full cars and 9 lb. per ton for empty cars. Also, from this paper, was taken the reduction of 13.7% in the tractive power for the speed of 10 miles per hour.

The hauls in Table 1 are from the records of The Pennsylvania Railroad Company, and represent a fair average of actual train weight for one locomotive of the type described, for a speed of 10 miles per hour.

TABLE 1.

Compensated grade. Percentage.	Gross train weight in tons.
0.300	3 292
0.655	1 898
1.055	1 238
1.260	953
2.130	555

Using the values in Table 1, and increasing the train capacity 10% for overload, as a limiting maximum, the coefficient of tractive

* *Transactions*, Am. Soc. C. E., Vol. L, p. 1.

power was established as 0.232; and the complete expression for tractive power then became 0.232 $\times$ 0.863 $\times$ the number of pounds on the drivers, or 0.2 $\times$ the number of pounds on the drivers, in which the 0.863 is the complement of 13.7 per cent. Introducing now the factor for train resistance, and denoting the ruling grade percentage by g, the general equation is obtained:

$$\left.\begin{matrix}\text{Tons of}\\ \text{train weight}\\ \text{behind tender.}\end{matrix}\right\} = \frac{0.2 \times \text{pounds on drivers.}}{9 - 5\,\dfrac{\text{Weight loaded on train.}}{1.1 \times \text{Capacity of train.}} + 20\,g} - \left\{\begin{matrix}\text{Tons of}\\ \text{locomotive}\\ \text{and tender.}\end{matrix}\right\} \quad . . (1)$$

in which the "Capacity of train" is the sum of the capacities of the cars, and all tons are 2 000 lb. in weight. For the particular case in hand, the weight on drivers is 173 000 lb., and the weight of locomotive and tender is 168 tons; hence, with cars of any class loaded to the maximum, the equation assumes the special form:

$$\text{Net tons behind tender} = \frac{34\,600}{4 + 20\,g} - 168.$$

Using this equation and the values of g for the hauls given above, from which the coefficient of tractive power was derived, it is found to give the train weights within about the weight of one empty car, except in the case of the fourth haul, which it makes 64 tons too high; but, as the last haul is only 19 tons too high, it is reasonable to assume that the equation represents safe values in the case of this example. Also, it is reasonable to assume that since for empty cars the train resistance is 9 lb. per ton of train weight, the values intermediate between 4 and 9 lb. per ton will be given correctly by the equation for all partial ladings of the train. It will be noted that for a train of empty cars the equation becomes:

$$\text{Net tons behind tender} = \frac{34\,600}{9 + 20\,g} - 168.$$

Because the purposes for which this equation was derived are usually expressed in terms involving an annual paying tonnage to be moved, the form of the general equation is believed to present special advantages, and from it a table may be formed showing the tons of lading and the number of cars in the train, which is convenient in estimating operating expenses; and the possibility of using it in making up trains is also evident.

The equation has thus far fulfilled the requirements for which it was designed, but suggestions have been made which show that it can be greatly improved in point of accuracy of form and adaptability in making up trains.

One objection to the form is found in the fact that the car resistance per ton varies in a straight line, or, in other words, that it is proportional to a constant increment of lading. Every indication in Nature goes to prove that probably the variation should be in proportion to an increment of lading which is constantly increasing, or to a decrement of similar character; in short, it should represent a curve instead of a straight line when depicted graphically. The grade resistance, 20 g, is evidently mathematically correct, being a simple case of a weight on an inclined plane; but the further suggestion of reducing the value of the tractive power for the grade involved with the dead weight of locomotive, tender, and caboose, has a great advantage in point of accuracy over the method of simply subtracting such dead weight as indicated in the above equation, particularly in cases where the tractive power is determined by a dynamometer car. For general use, it seems advisable to express the relation of the train weight to the number of cars as the average total car weight. It can readily be seen that if the four axles and eight wheels of an ordinary car are identical in size and weight, then the total weight of the train divided by the number of cars is an average car weight, which is a true measure of the resistance per car. If the condition is not realized absolutely, the average car weight will be affected by an inconsiderable amount in the case of an ordinary mixed train.

In view of these considerations, the equation must necessarily be expressed in a form somewhat different from that given above, and, for this purpose, the writer makes use of the following values, which are selected from a number of tests made with a dynamometer car, in which R is the resistance of a train, in pounds per car, and W is the average weight per car, in tons of 2 000 lb., for a speed of 10 miles per hour:

TABLE 2.

Point.	R (pounds).	W (tons).
A	160	20
x	195.7	40.2
y	199.0	42.8
B	216.0	72

Referring to Fig. 1, upon which the points, A and B, are located, it is clear that the resistance per car will decrease from B to A, and will become zero when the number of cars becomes zero. Hence the line of variation must pass from B through A and through the origin

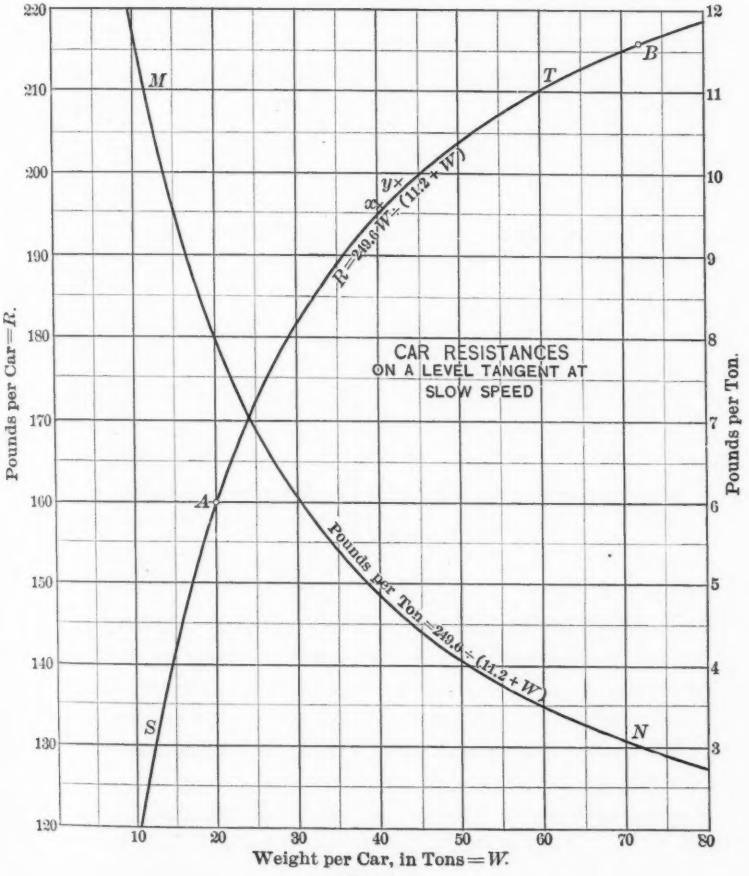

FIG. 1.

of co-ordinates, zero. It must, therefore, be a curve; and such a curve is represented by the equation:

$$R = \frac{a\,W}{b + W};$$

its particular form for this case being,

$$R = \frac{249.6\,W}{11.2 + W},$$

which is the line, $S\ T$, in Fig. 1. Plotting the points, x and y, it is seen that they agree closely with the values given by the equation, thus confirming the assumption that the resistance per car varies as a curved line of this character.

Dividing the values of R by the corresponding values of W, and plotting the results to the scale on the right of Fig. 1, develops the curve, $M\ N$, of pounds per ton of train weight.

It is now possible to formulate the equation:

$$\text{Tons of train weight behind tender} = \frac{P - 20\ g\ m}{\frac{a}{b + W} + 20\ g} \quad \ldots\ldots\ldots (2)$$

in which P = the pounds of tractive power at the drawbar; m = the weight, in tons, of the locomotive and tender; a and b are constants; W is the average weight per car, in tons; and g is the grade, in percentage. The tractive power, P, is here reduced by 20 g m (where the weight of a caboose may be included in m if desired), instead of subtracting the dead weight, m, after the division has been made. Substituting the known terms taken from Tables 1 and 2 and those derived for Fig. 1, an average value of P is found to be 32 100 lb., and therefore the equation for train weight for this particular case becomes:

$$\text{Tons of train weight behind tender} = \frac{32\,100 - 3\,360\ g}{\frac{249.6}{11.2 + W} + 20\ g}.$$

This equation does not agree with the values in Table 1 as closely as do the results from Equation 1, but it is more rational in form, and is adapted to use in connection with records taken with a dynamometer car, from which the tractive power, P, and the constants, a and b, can be readily determined by experimental trials for any particular forms of locomotives or cars.

For the purpose of making up trains, an equation similar to the above can be expanded into a table. Thus, for any particular division of the railroad, the ruling grade, g, is known; assuming it to be 0.5%, for example, the equation becomes:

$$\text{Tons of train weight} = \frac{30\,420}{\frac{249.6}{11.2 + W} + 10}.$$

From this may be prepared for all car weights a form similar to Table 3, which shows only three values of average car weight. The

reduction for temperature, of course, must be determined by past experience. A similar table, prepared for each class of locomotive on the division, can be used for making up trains by a yard-master of very mediocre intelligence. The process of taking the continued sum of the car weights until the total indicated in the table is approximately reached, and then dividing the result by the number of cars to obtain the average car weight, is a simple one. The difference of a few cars more or less is then made up, and the total train weight of the completed train is finally checked by a repetition of the process.

TABLE 3.

Average car weight, in tons.	M. & N. Division—Locomotive Class H.				
	Train Weights for Various Temperatures.				
	Summer.	45° — 25°	25° — 5°	Below 5°	Emergency.
40	2 046	1 840	1 636	1 535	1 023
41	2 058	1 852	1 646	1 544	1 029
42	2 071	1 864	1 657	1 553	1 036

This example of the manner in which trains may be made up is given to illustrate the practicability of dispensing with a "car factor," consisting of a predetermined number of tons to be added to each car in the train. Such a car factor is in reality a fictitious weight, and its use introduces the element of a "straight-line variation" in reaching the final result, which is questionable in point of accuracy. The following quotation is taken from "Tonnage Rating," an article by Mr. F. W. Thomas*:

Mr. Thomas says:

"The most difficult problem after the rating has been ascertained and proven, is to express the rating intelligently and in such a form that the dispatchers, yard-masters, foremen or switch crews and conductors can understand and easily interpret the rating sheets.

"The most difficult thing to impress upon those interested is the fact that the rating is often governed by the number of cars in the train; the greater the number of cars, the greater the rolling resistance. In the eyes of the average train master and dispatcher a thousand tons, whether confined in twenty cars or in fifty, is a thousand tons."

* * * * * * *

* *American Engineer and Railroad Journal*, April, 1907.

"You will note * * * that the maximum rating is shown in cars weighing fifty tons, and for every car added to the train above this given number of cars a reduction from the maximum rating must be made of from four to five tons. * * * I cannot say that this reduction is based on any fixed rule, beyond, as mentioned above, that it is the fruit of long investigation and a series of tests * * * ."

The writer thinks that Mr. Thomas intended to say, in the instance above, "the greater the number of cars the greater the rolling resistance per ton of train weight," which is usually difficult to make clear.

As a matter of fact, the determination of the car factor is sometimes based on a "fixed rule," or rather a carefully wrought out series of experimental tests put into a rational mathematical form. This form usually is developed by equations, some of which represent straight lines, but whether or not this be true, the unfortunate fact remains that the use of such a car factor, when derived, constitutes a constant increment, thus introducing a form of straight-line variation into the calculation.

It may be asked, what objection can be made to such a variation, and in answer the writer appeals to the reason of those who are students of the problem here presented. Natural forces represented graphically, almost without exception, are forms of curves. The construction of a train is artificial, but the forces developed by the movement of the train are natural in their characteristics; and continued study brings with it the growing conviction that the graphics in this problem should be curves rather than straight lines. Granting this, the use of the car factor must be dispensed with, and a rigidly curvilinear method or equation must be formulated. If we conceive of a level railroad tangent as an arc of 4 000 miles radius, the center of which is the earth's center, then Equation 2, as given above, is strictly curvilinear in its components and in its entirety; but if this conception appears to violate the sense in which the quantity "20 g" is used, there still remains the argument that grade is an artificial and not a natural element of force in this problem.

The economic issues dependent upon a true determination of maximum train weights, especially for slow freight trains, are of such grave importance that the writer feels justified in stating that any criticisms or suggestions which will throw more light on the subject will be welcomed by the majority of railroad men.

The data upon which the foregoing developments are based are believed to represent normal conditions. They are the results of many tests, and of careful selection and reduction. In every case, however, they apply only to the slow speed of from 10 to 7 miles per hour. The equations are given to show only the form of variation, since it is apparent that the constants in them must be derived from experimental tests.

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PAPERS AND DISCUSSIONS

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THE PURIFICATION OF GROUND-WATERS CONTAINING IRON AND MANGANESE.

Discussion.*

BY MESSRS. W. C. LOUNSBURY, ARTHUR L. TERRY, JR., PHILIP BURGESS, A. ELLIOTT KIMBERLY, AND ALEXANDER POTTER.

Mr. Lounsbury.

WILLIAM C. LOUNSBURY, ESQ.† (by letter).—Mr. Weston mentions the difficulties which may be encountered in removing iron from underground supplies. The plant at Superior, Wis., has certainly had its share of troubles arising from this difficulty, and a brief account of the somewhat peculiar conditions and experiences may prove of interest, in the light of Mr. Weston's suggestions.

Superior is the second city in Wisconsin, and is located at the head of the Great Lakes. A narrow strip of sand, some 7 miles long and a few hundred feet wide, runs parallel to the Wisconsin shore, about a mile from the city and mainland. Lake Superior washes the outer shore of this sandy stretch, and Superior Bay, the inner shore. This land, called Minnesota Point, is principally made up of a fairly coarse sand, mixed with considerable quantities of fine particles of metallic iron. In the formation of the point, logs, peaty matter, clay, and débris have undoubtedly been incorporated with the sand, so that the clean sand of the surface shows no indication of what may be underneath.

About ten years ago, the Water Company decided to draw its supply from wells to be located on Minnesota Point, along the shore of the lake. By this means, it was thought, an excellent supply of

* This discussion (of the paper by Robert Spurr Weston, Assoc. M. Am. Soc. C. E., printed in *Proceedings* for December, 1908), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Chemist and Superintendent of the Water-Works, Superior, Wis.

filtered water could be obtained. Accordingly, eighty odd wells were jetted to a depth of about 14 ft. These wells were 6-in. wrought-iron pipes, carrying Cook brass strainers, and were spaced about 17 ft. apart, each being separated from the main suction by a valve. The tops of the wells were below high-water level, and the ingress of the water was some 25 ft. below this. The shore line varies, but the wells are approximately 20 ft. from the lake and 400 ft. from the bay across the point. During high water, the water washes over the wells, but, on account of the ice in winter, and in times of low water, the free water of the lake may be several hundred feet away. The water derived from these wells was extremely disappointing. It was harder than the lake water, although not as hard as the bay water; moreover, it carried considerable iron. Mr. Lounsbury.

In 1901, G. C. Whipple, M. Am. Soc. C. E., made an analysis of the water, as follows:

Total solids	78

Nitrogen as:	Total.
Albuminoid ammonia	0.066
Free ammonia	0.016
Nitrites	0.001
Nitrates	0.04
* * * * * *	
Iron	1.50
Hardness	48.5
Alkalinity	46

The character of the water has remained practically unchanged since. The water as described was used under direct pressure to the city for several years, but caused much trouble. The iron settled out in the pipes; *Crenothrix* appeared in great quantities; and all the characteristic objections to iron manifested themselves in a marked degree. The water was particularly disagreeable because it became highly colored, after being drawn from the wells, and this highly-colored water was confounded, by the public, with the objectionable, highly-colored water of the bay, under which the intake passes. This latter water, it may be stated, carries considerable organic matter, leached from tamarack swamps, and also iron in solution. Its color varies above 100.

In 1899, Allen Hazen, M. Am. Soc. C. E., designed and constructed an iron removal plant, for the local company. This plant* consists, essentially, of an aerating device, by which the water showers through holes in iron trays, and falls as spray into a well beneath. The trays are enclosed in a skeleton house, the sides of which are

* Described at length in *Engineering News*, February 21st, 1901, p. 141.

Mr. Lounsbury.

shutters, arranged so as to give free access of the air to the water. From here, the water flows to covered sand filters, and is filtered at a rate of about 5 000 000 gal. per acre, per day. This method of treatment at once gave improved conditions, and the effluent has never since supported any growth in the mains, or deposited iron in them to any extent. Strangely enough, however, the water showed, by analysis, an iron content of 0.7, or 0.8, or even 1.0 part of iron in the filtered water, and an average color of 30, varying from a minimum of 25, to a maximum of 40. To reduce the iron still further, the aerator was heightened, so that more air came in contact with the water. This gave no beneficial results.

The carbon dioxide content is fairly constant, at 1.2 parts. Aeration as used reduces this to an amount of from 0.4 to 0.6 parts.

The writer's experience with the plant commenced in May, 1906. It was very important, at that time, to remove the color of the effluent, on account of local political conditions. In studying the matter, and in the light of a few bottle experiments, the plant was run for several days without the use of the aerator. The result was an immediate reduction of color and the iron content in the effluent. The iron was almost completely precipitated, and the color was reduced so much that the water was perfectly satisfactory. The results are indicated on the diagram, Fig. 9, which shows the decided change that takes place when the aerator is put into use, or is by-passed.

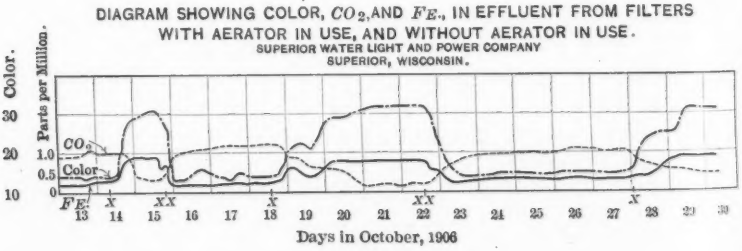

FIG. 9.

In Fig. 9, the time of putting the aerator into operation is marked with a cross, *X*, and the time of by-passing is marked with a double cross, *XX*.

The samples of water were collected at the pumps, so that the effect is seen several hours after the time marked. Samples were analyzed at least once in 4 hours, during the entire period shown. The slight increase of color on October 16th is due to placing in operation a filter previously filled with aerated water.

The analyses were all made by the writer, and are expressed according to the American Public Health Association standards.

The most obvious thing done by this treatment, or lack of treatment, was to allow the filtration to take place in the presence of a greater amount of carbon dioxide, and to pass the water to the filters without violent shaking. There is always an excess of air in the water, through leakage of the system and at the pumps, therefore the writer is inclined to think that the amount of oxygen present does not play an important part. This condition was exactly what had been found in the laboratory. Bottles of the same raw water had been taken. Into one was run carbon dioxide, charging the water under the atmospheric conditions prevailing. The other bottle was simply stoppered. Both were well shaken, and allowed to stand. It was found that the one containing the excess of carbon dioxide always precipitated its iron readily, without attaining the yellow color the control bottle did. Other bottles, after having stood a year or more, have invariably deposited most of the iron, although not always losing their color. The quantities of precipitant deposited on the filters when operating without aeration, was so great that the filters were totally inadequate to handle it. Ordinarily, with aeration, a filter runs from 7 to 9 days; without aeration, one dies in 3 or 4 days. Mr. Lounsbury.

Non-aeration has been tried, at different times, for more than two years, and has invariably produced a satisfactory effluent, having a color of from 15 to 18 and an iron content of never more than 0.5 part, and frequently as low as 0.3. These experiments have covered ranges of water temperature from 34 to 60°, and air temperature of from —30° to +90°, the rates of filtration varying with the varying consumption of the city, and the loss of head due to the age of the filters. A proportion of lake water has also been admitted, the alkalinity of which was 40, and the hardness 37, and containing no iron; but always with the same result.

Mr. Weston reports a manganese content of 0.12 part in this water, and possibly this plays an important part in the trouble. Whatever the explanation may be, it is certain that the Superior water has been so constant in composition for the past few years that the iron can be readily and easily precipitated, and the color can be kept from forming, by filtering without aeration. Possibly Mr. Weston can explain just why this is so.

ARTHUR L. TERRY, JR., JUN. AM. SOC. C. E. (by letter).—It may be of interest to those undertaking the purification of ground-water to know that a deferrization plant of 700 000 gal. daily capacity was installed for the Government by Hungerford & Terry at Fort Hancock, Sandy Hook, N. J., more than a year ago. Mr. Terry.

The original water supply of the fort consisted of a series of well points driven into the sand about 6 ft. below the surface. These gave a very good water until the consumption reached about 75 000 gal. a day, when evidences of sea-water began to be found, and increased to

Mr. Terry. such an extent that four years ago two deep wells were installed, one 450 ft. and the other 750 ft. deep. The water from both these wells was very soft and good in every way, except for the iron content, which reached about six parts per million in both wells. The deeper well flowed at an elevation of 15 ft. above tidal water, discharging about 20 gal. per min. The water contained considerable quantities of hydrogen sulphide. Like most deep-well waters, it was clear when first discharged, but in a short time developed considerable turbidity. An attempt to use this water without treatment resulted in so much dissatisfaction that the authorities reverted to the old supply. After a year, two settling reservoirs were built and air-lifts were installed, the idea being that aerating the water would oxidize the iron; and that the settling reservoirs, which gave about 5 days' sedimentation, would permit all the iron to be removed. This plant was completed about 2½ years ago, and put into operation, but it was found that while the oxidation of the iron appeared to be very rapid, and that this precipitated iron settled with considerable rapidity, yet about 30% of the iron remained in solution. At about that time a proposition was made by Hungerford & Terry to install a plant using lime as a coagulant, utilizing the settling reservoirs and supplementing the plant with mechanical filters. This proposition was accepted, and the plant was installed substantially as follows:

The two wells, each of which would discharge 250 gal. per min. when using the air-lift, were conducted into an orifice box containing two adjustable orifices. The large orifice is set so that the head of water varies from 1 to 2 ft. over the orifices, depending on the discharge of the wells. The area of the large orifice is about 17 sq. in., that of the small orifice is about ¼ sq. in., and both are placed at the same elevation, so that the discharge of one is approximately proportional to that of the other, regardless of the fluctuations in head. The small orifice discharges into a lime saturator. This saturator consists of a conical vessel, at the bottom of which is placed about a bushel of freshly slaked lime. The incoming water from the small orifice is discharged at the bottom of the saturator, and, to aid in a thorough mixing of the applied water with the lime, a small air line is run from the air compressor, which produces a constant ebullition part way up the saturator. A conical hood, terminating in a pipe reaching above the water level, carries the air to the surface after it has performed its duty of mixing the lime and water. All below this hood is a violently agitated milk of lime, while above and outside the hood there is a solution of lime gradually clearing as the surface is reached. The outlet of the saturator is on the side at the top, and when raw water enters the saturator it displaces an equivalent amount of a clear saturated solution of lime, and in its turn is converted to milk of lime. The quantity of water applied to the saturator is ordinarily

about 7 gal. per min., and this, of course, displaces a corresponding quantity of lime water. This lime water is applied to the raw water from the large orifice as it enters No. 1 reservoir. Coagulation is almost instantaneous, and is complete. Mr. Terry.

The water takes about two days to pass through the two settling basins, and, although apparently clear, a very considerable amount of iron is still left. This fact has been determined, not by analysis, but by observing the wash-water from the filters, the latter requiring washing twice a week.

The filters are mechanical filters of the pressure type, each having a capacity of 360 000 gal. per day of 24 hours, or 720 000 gal. per day for the two units.

The lime required for this plant costs only 80 cents per million gallons, and does not seem to vary with the seasons, for the lime feed has not been changed since the plant was first installed. That the correct quantity of lime was being used was determined by testing the alkalinity of the water with phenolphthalein. When the filtered water shows a faint pink color with phenolphthalein, the removal of the iron is almost complete, although a pink reaction in the water before filtration also indicates very satisfactory results. It is a rather remarkable fact that the water reacts strongly to phenolphthalein before filtration, and yet shows no reaction afterward.

The following report of the water comes from the office of the Surgeon-General of the United States Army:

"The first [sample] is the water as it comes from the wells. It is a turbid water of a very faintly alkaline reaction, odorless, of a yellowish color, due to the marked amount of suspended ferruginous matter. This settles on standing to an appreciable deposit, leaving it a clear, colorless supernatant fluid.

"The second is the water after treatment through the filtration apparatus. It is absolutely clear and colorless, is faintly alkaline in reaction, contains an almost inappreciable sediment on standing for many hours, and is without odor. There is no visible charring or carbonization in the residue on its incineration. There thus appears a great improvement in the water by the removal of what small amount of organic matter it contained, together with practically all of the iron, and at the same time the amount of the mineral matter was reduced in its total by one part per million. The actual amount of iron present in this sample is 0.0140 part per hundred thousand, while its hardness (Clark's) is 4.54 per hundred thousand.

"EDWIN R. HODGE,
"Chemist in charge."

Unfortunately, the Government did not report the analysis of the raw water as it came from the wells, but another analysis, made at the same time, showed the total iron to be six parts per million.

Before the receipt of the analysis of this sample, the contractors

Mr. Terry. requested that another sample be taken by the Government for analysis, they believing that a still higher efficiency could be shown by increasing the lime to such a point that the filtered water would react slightly with phenolphthalein. This was forwarded to the War Department, and the following analysis was returned:

"Contained iron 0.0088 part per hundred thousand.

"Total solids: Treated water, 49.50. Raw water, 82.50 parts per million.

"EDWIN R. HODGE,
"Chemist in charge."

Experience with this plant indicates that the lime treatment, when installed as described, is cheaper than any other process, not excepting aeration. To be sure, an air-lift is used in conjunction with this plant, but that is only for 6 hours per day, and, during the remaining time, the well flows and there is no aeration of the incoming water, the result being obtained by the lime alone.

When it is considered that this plant has no skilled supervision, but is run by employees whose sole duty (with regard to the plant) is to wash the filters twice a week, and slake and discharge into the saturator a few bushels of lime; and, also, that the plant always maintains an almost total reduction of the iron content, this method of deferrization is worthy of notice. Those who have tried to introduce clay or milk of lime into water with any degree of accuracy know what a difficult and unsatisfactory proceeding that is.

Taking the experience at Reading, reports state that the water was hardened very considerably. On the other hand, the Fort Hancock plant showed virtually no change in the hardness, except on the occasion of the second analysis, when the lime was increased to a point at which softening began; but then, instead of increasing the total content of the water, there was a reduction of nearly 50% in the total solids.

Referring to aerating towers of coke or other substances, it seems, from the knowledge at hand, that elevating the water and cleaning such towers, to say nothing of the interest on the investment, would make the cost greater than treatment by lime, and there can be no doubt that a lime-treated water will contain far less iron than one which relies merely on aeration.

To carry the matter further, it even seems that some of the abnormal conditions which have arisen at Posen and other places might be handled successfully by the lime process, followed by subsequent filtration. A slight excess of lime applied to a water containing a large amount of humus is a most energetic coagulant, and it certainly should be far more effective than the present methods.

The only objection that the writer can see to the use of lime is in the uncertain methods by which it has been applied in the past. An

excess of lime, under ordinary conditions, is an unpleasant feature, but when applied in the manner described, an excess is an impossibility if the plant is designed properly. The worst that could occur would be a deficiency due to inattention of the operative in not placing slaked lime in the saturator. Mr. Terry.

PHILIP BURGESS, ASSOC. M. AM. SOC. C. E. (by letter).—This excellent paper will doubtless prove of great interest to all engineers engaged in the installation and operation of public water supplies. The information contained tends to make clear many of the problems encountered in the search for an economical water supply of satisfactory quality. The question of approval of ground-waters has been frequently referred to the Ohio State Board of Health, and the writer has made many examinations of water from ground sources, waters which have contained frequently such large quantities of iron as to be unsatisfactory for domestic use without purification. Mr. Burgess.

In the Middle West it is frequently necessary for the engineer to judge between the advantages of a supply from ground sources and one of purified surface-water. While the former is frequently clear and potable, it is sometimes so objectionably hard that a surface-water would be more desirable. The double cost of pumping, generally necessary with ground-water supplies, sometimes makes the cost of operation less than that of a filtered surface-water only to the extent of the cost for a coagulant necessary when the water is purified with mechanical filters. The costs of construction shown by Mr. Weston in Fig. 1 are evidently based on slow sand filters. The average cost of thirteen mechanical filter plants of varying capacities in Ohio, including low-service pumps, coagulation and sedimentation basins, filters, and clear wells, is about $14 000 per million gallons, while the cost of operating the purification devices averages about $6 per million gallons. These figures show that the cost of securing a filtered water supply of surface origin is frequently not much greater than that necessary to procure a satisfactory ground-water. Moreover, the latter is frequently so hard that the total expense to the consumer is considerably greater than is the case with the softer surface-water. Cases are not infrequent where a ground-water supply has become dangerously polluted without the knowledge of the operator until a considerable quantity of badly polluted water has been pumped to the consumers. In the writer's opinion, such conditions are very rare with a well-constructed and operated filter plant.

One feature relative to the distribution of iron-bearing waters is little touched upon by Mr. Weston, and that is the great variation in the quantity of iron in water from different strata in the same locality. The writer has in mind, especially, two wells at Newton Falls, Ohio, one 58 ft. deep containing 3.0 parts of iron per million, and the other, not 50 ft. distant, 35 ft. deep, and containing 20.0 parts of iron per

Mr. Burgess.

million. Similar variations have been found at different depths in other parts of Ohio.

Mr. Weston's discussion on the design of iron-removal plants is so general that it will hardly serve as a reference for engineers who have to study the problem of the purification of a specific ground-water. The scarcity of the data on such plants in America and the great variations in the character of the water to be treated, doubtless account for the absence of more specific statements. The following data are presented in order to show what has actually been accomplished by iron-removal plants at Garrettsville and at Shelby, Ohio.

The water supply for Garrettsville is derived from ten driven wells varying in depth from 41 to 70 ft.; also from a shallow dug well or flowing spring. The wells and spring are in a ravine just above the confluence of two small streams. The wells pierce, first, a layer of yellow clay of varying depth, and second, a layer of white sandstone rock, from 40 to 60 ft. in depth and of nearly pure silicious material. The lower 8 ft. of the sandstone strata are of a conglomerate nature and comprise the water-bearing material from which all the wells derive their supply. The water is of generally satisfactory quality for domestic use, except that its iron content is about 2.0 parts per million. It was for the removal of iron that a purification plant was constructed in 1906. The devices provided for the removal of iron comprise an aerator, a sedimentation basin, and a mechanical filter.

The aerator consists of an umbrella-shaped device on the discharge of a low-service force main. It is 4 ft. in diameter and approximately 5 ft. above the level of the water in the basin. When the plant is in operation, water from the wells is lifted by the low-service pump, and, at the point of discharge into the basin, falls back on this umbrella-shaped device which breaks it up into a fine spray. The sedimentation basin is divided into four compartments, about 12 ft. square and 9 ft. deep. Its total capacity is 39 000 gal. The filter is of the gravity mechanical type, and is a rectangular concrete tank with a total area of 96 sq. ft. The filtering material comprises 12 in. of graded gravel and 36 in. of sand of about 0.3 mm. effective size.

During a recent inspection by the writer, the consumption at Garrettsville was about 25 000 gal. daily and the rate of pumping on low service about 2 000 gal. per hour. This gives a period of sedimentation of practically 20 hours and a rate of filtration of approximately 21 000 000 gal. per acre per day. The settling basin is cleaned about once every 3 months, and it is found that practically all the sludge accumulates in the first compartment, where there is generally a deposit about 1.5 in. in depth. Every 3 days it is necessary to back-flush the filter, which is accomplished by applying filtered water from the reservoir. Washing is accomplished in about 10 min., and requires about 15 000 gal. of water, equivalent to about 20% of the total quantity filtered.

Mr. Burgess.

The consumers appear to be very well satisfied with the quality of the water and the plant is operated without difficulty, but at considerable excess cost due to the unusually large quantity of wash-water necessary. This could be reduced by improving the washing system of the filter.

The water supply for Shelby, Ohio, is obtained from ten 6-in. driven wells in the alluvial plain bordering Black Fork Creek. The wells vary in depth from 40 to 60 ft., and are from 200 to 300 ft. from the creek. They pierce the following material, in the order stated: Loam, clay, hardpan, gravel, slate, sandstone rock, and water-bearing gravel. The thickness of the strata is not known, but it is said that the slate and sandstone are both thin. The fact that leaves and twigs were found in some of the wells at considerable depths indicates that, in prehistoric times, the ground from which the water is obtained was of a swampy character.

During pumping, water generally stands in the wells about 19 ft. below the surface of the ground, and it was not lowered during the recent exceptionally dry spell. This would indicate that the water is derived from ground sources, and not from the creek. Although objectionably hard, the water is of generally satisfactory quality for domestic use, except that its iron content is 6.6 parts per million.

The water was used without purification for eight years, but considerable trouble was caused by the precipitation of the iron in the mains, and a purification plant was built in 1905. This plant comprises an aerating tank and two horizontal pressure filters. The aerating tank is of steel, 120 ft. high, and 8 ft. in diameter, and is placed concentrically within the old stand-pipe, which is 130 ft. high and 16 ft. in diameter. In the top of the tank are provided steel plates perforated with 1-in. holes 2 in. apart. The plates are 2 ft. apart, staggered, and each covers one-half the cross-section of the tank, so that the actual distance between the plates is 4 ft.

The filters are horizontal steel tanks, 24 ft. long and 8 ft. in diameter. In the bottom is a collecting system, above which is placed 18 in. of screened gravel and 18 in. of crushed quartz sand. The latter will all pass through a screen of 16 meshes per inch and be retained on a screen of 30 meshes per inch. Its effective size is approximately 0.75 mm.

The present daily consumption at Shelby varies from 250 000 to 300 000 gal. Water from the filters is pumped directly into the mains, and the plant is operated 24 hours. The force main from the low-service pump is carried up inside the outer compartment of the stand-pipe and discharges into the aerating tank, in which the water is generally maintained at least 10 ft. below the top. The aerated water flows by gravity through the filters, and is pumped into the mains. The average period of sedimentation in the tanks is about 4 hours, and the filters are operated at about 30 000 000 gal. per acre per day.

Mr. Burgess.

They are washed once a week, using from 20 000 to 40 000 gal. of water. The initial and maximum losses of head are 2 and 6 lb., respectively. The stand-pipe is flushed out about once a month, but little sediment is removed.

On January 14th, 1909, the writer, in company with Mr. C. F. Long, of the Laboratory Department, made an examination of the Shelby plant. During this examination it was the purpose, as far as possible, to make the necessary analyses in the field. The temperature, turbidity, color, dissolved oxygen, alkalinity, iron, oxygen consumed, and total bacterial content were determined at the plant, while the other analyses shown in Table 19 were determined on samples shipped to the laboratory of the State Board of Health. The examination covered 18 hours' operation of the plant, but indicated little variation in the character of the water at the several sampling points.

TABLE 19.—RESULTS OF ANALYSES OF SAMPLES COLLECTED AT SHELBY, OHIO, ON JANUARY 14TH, 1909.
(Parts per Million.)

Determinations.	Raw water.	Aerated water.	Filtered Water.	
			At filter.	In mains.
Temperature, in degrees, centigrade	10.9	10.1	9.9	11.5
Turbidity	0+	60±	10±	5+
Color*	3	.....	45	25
Free CO_2	48	28	24.5	22
Dissolved oxygen				
(Parts per million)	0	7.8	7.7	7.5
Percentage saturated	0	69	68	69
Alkalinity	263	248	241	240
Incrustants	214	220	215	220
Total hardness	477	468	455	460
Total solids	713	.....	713	.....
Loss on ignition	73	.....	82	.....
Total iron (Fe)	6.5	6.7	0.85	0.60
Magnesium	31	.....	31	.....
Chlorine	11.8	12.5	11.5	13.5
Oxygen consumed	3.65	2.58	2.13	1.90
Nitrogen as				
Free NH_3	5.80	.....	0.180	.....
Alb. NH_3	0.112	.....	0.104	.....
Nitrites	0.002	.....	0.002	.....
Nitrates	0	.....	0	.....
Bacteria per cubic centimeter	1	.....	1	.....

* On unfiltered samples.

The analyses of the raw water indicate that it was clear, hard, of high iron content, and contained no dissolved oxygen; its free CO_2, free ammonia, and oxygen consumed were all high, while nitrates were absent. The water was practically sterile.

The analyses of the aerated water indicate a considerable change by aeration, notably the increase in turbidity due to the oxidized iron; the elimination of considerable free CO_2 displaced by dissolved oxygen; and a reduction in alkalinity. The total iron content was not reduced,

but the latter was in a changed form, and was chiefly in suspension as an oxide. Mr. Burgess.

The analyses of the filtered water indicate that the filters removed most of the suspended iron, so that the final product contained but 0.85 part per million. The color determinations indicate that the residual iron was in suspension. The samples from the mains indicate a further reduction, or iron sufficient to cause an accumulation of deposits in the mains and a slight staining of plumbing fixtures.

In the writer's opinion, the efficiency of the filters could be considerably improved were the filtering material replaced by a greater depth of finer-grained sand having an effective size of about 0.3 mm. In general it may be said, however, that the purification devices have served to eliminate the objectionable features so that consumers are now generally satisfied with the quality of the water.

An interesting point brought out by the analyses is the considerable reduction of alkalinity by the aeration of the water and the oxidation of the iron, a fact which tends to show that the latter is present in the form of a carbonate. There is also a striking reduction of free ammonia, which apparently goes off as a gas, as the filtered water contains no nitrates. Further information on these points is very desirable.

A. Elliott Kimberly, Esq.* (by letter).—While there are many factors which favor the use of water supplies from ground-water sources, it is to be remembered that ground-waters in the Middle West are usually hard and, from this standpoint, objectionable. Methods of water filtration for the removal of suspended matters and bacteria have reached such a state of efficiency that, ordinarily, it is feasible and economical to consider thoroughly the use of a surface-supply, rather than derive the supply from ground-water sources. This is particularly true where the surface-water is soft and the ground-water hard. The small increase in operating charges incidental to the operation of a filter plant is but slightly more than the actual money loss incurred by the consumer of the hard ground-water. The argument is still more forcible where deferrization is necessary, and, before deciding on the use of a hard, ferruginous ground-water, all reasonable means should be investigated for obtaining a softer supply from surface sources. Mr. Kimberly.

The presence of iron in ground-waters often prevents their use as sources of water supply, unless means be provided for removing the iron, and to this important subject Mr. Weston has certainly contributed a mass of valuable data.

The writer was at one time engaged in studying the water at Reading, Mass., under the direction of Mr. H. W. Clark, where the difficulty encountered in removing the iron was attributed, not so

* Asst. Engr., Ohio State Board of Health.

Mr. Kimberly.

much to the presence of organic matter, as to the fact that the iron was present in the form of a sulphate, or, more correctly, was accompanied by the sulphate radical, SO_4, rather than by the carbonate radical, CO_3. To this fact was ascribed the greater ease of iron removal by aeration possible in the cases of the Provincetown and the Watertown waters.

It is interesting to note that it is not the nature of the acid radical, but the presence of organic matter, as Mr. Weston's researches clearly show, which prevents the precipitation of the iron after it has been oxidized.

As is well known, the removal of clayey matters in suspension is readily accomplished in water-filtration practice by sulphate of alumina. The use of a clay-bearing, ferruginous water as a precipitant for domestic sewage, as at Hanley, England, suggests several interesting Ohio experiences.

The sewage of Sebring, Ohio, in volume about 46 000 gal. in 24 hours, flows in an open ditch into Sulphur Creek, which has its rise in an abandoned coal mine. In dry weather, the creek flow is at the rate of about 28 000 gal. in 24 hours. The crude sewage is essentially domestic, is ordinarily weak, and contains a predominance of sink wastes and soapy matters. The Sulphur Creek water is an acid mine waste which contains free mineral acid, very large quantities of iron, high sulphates, and other constitutents peculiar to mine drainage. Analyses of the Sebring sewage and of the Sulphur Creek water are given in Table 20.

About 150 ft. below the confluence of the two streams, marked coagulation is visible. The velocity of the current is very slight, and, for a mile or more from the point of confluence, the stream is practically a sedimentation basin under continuous operation. At its outlet, one mile below the confluence, the supernatant water shows practically no turbidity and, besides being substantially free from iron and free acid, contains but little evidence of the marked organic pollution it has received.

The bottom of the stream, however, is a mass of putrefying sludge with a slight surface coating of ferric hydrate, surmounting a mass of black, nauseating, and foul deposit. These conditions are illustrated in Table 20. They indicate the effect of semi-colloidal sewage upon the iron present in mine wastes, and are another example of the deferrization of ground-waters by means of colloids.

Several further illustrations of the use of negatively-charged clay particles in hastening coagulation occur to the writer. At Warren, Ohio,* the Mahoning River water is purified by mechanical filters of the wooden tub type, and, at times, the turbidity of the raw water is so slight that clay is used to start the coagulation. At Newark, Ohio,*

* See forthcoming report of Ohio State Board of Health on Water and Sewage Purification Plants.

during certain seasons, the raw water is derived chiefly from the ground, and is so free from turbidity that it cannot be coagulated with sulphate of alumina unless clay be used. Clay has also been used at Harrisburg, Pa.,* and Ithaca, N. Y.,† to assist in obtaining a satisfactory floc under certain conditions of the raw water. Mr. Kimberly.

TABLE 20.—ANALYSES OF SEBRING SEWAGE AND SULPHUR CREEK ABOVE AND AT SEVERAL POINTS BELOW THEIR CONFLUENCE, ILLUSTRATING THE PRECIPITATION OF IRON-BEARING WATERS BY COLLOIDAL MATTER.

Sampling points.	Parts Per Million.							Bacteria per Cubic Centimeter.
	Turbidity.	Alkalinity.	Nitrogen as Free Ammonia.	Chlorine.	Dissolved Oxygen.	Iron (Fe).	SO_4	
Sulphur Creek.....	0+	—62*	0.8	23	0	50	1 044†	135
Sebring sewage.....	500	350	16.0	94	0	....	....	600 000
150 ft. below confluence.	250	154	13.0	59	0	40	808	8 000
1 mile below confluence.	5±	24	3.0	38	0	2	495	5 300

* Free sulphuric acid present. † Incrustants.

The question of iron removal is of special significance and importance in connection with problems of water supply for small villages. Recognizing the allowable limit of 0.5 part per million of iron in a potable water, what means are available and should be recommended for removing iron from ground-waters when present in objectionable quantities?

In reviewing the foreign and domestic deferrization plants, as described by Mr. Weston, it is apparent that there are several distinct types:

Aerators, sedimentation tanks and open filters (mechanical or slow sand);

Aerators and pressure filters;

Air-lift pumps, head tank, pressure filters, clear well or direct connections to mains.

Available evidence as to the best practice in the design of deferrization plants appears to be somewhat fragmentary, and it is apparent from Mr. Weston's paper that the design depends primarily on the character of the raw water, which should be carefully investigated. It is obvious that aerating devices and pressure filters which, in the absence of organic matter, are successful in removing the iron with

* Harrisburg, Report of Board of Commissioners of the Water and Lighting Department, 1907, p. 24.

† "Operating Results of the Water Purification Plant at Ithaca, New York," *The Engineering Record*, Vol. 57, 1908, p. 673.

Mr. Kimberly. sufficient thoroughness, show somewhat high operating costs, owing to the heavy loss of head necessary and the large volume of wash-water required. It would appear from Mr. Weston's theoretical discussion that, at times, aeration alone will not suffice to effect the precipitation of the oxidized iron, owing to the colloidal form of the ferric hydrate; nor will aeration, sedimentation, and open sand filters, when organic matter is present. It appears, therefore, that the only plan always feasible (except for water similar to that at Reading, Mass.) is a combination of aerators and pressure filters, or aerators, roughing filters, and pressure filters. In other words, the precipitation of the iron is effected ordinarily by a combination of chemical oxidation and mechanical separation induced by the adsorptive effect of the ferric hydrate previously precipitated upon the filtering medium. On this account roughing filters appear favorable as a means for lightening the load on the finishing filters, and doubtless would take the place of an aerating tower, particularly if the water be raised by an air-lift. In such cases pressure or open mechanical filters are to be considered, and not slow sand filters.

Mr. Potter. ALEXANDER POTTER, ASSOC. M. AM. SOC. C. E. (by letter).—The author has made an exhaustive and valuable research. The presence of iron and manganese in ground-waters is well established. The purification of such waters is an important subject, and has been treated fully in this paper. Although the author states that the paper "does not concern itself with methods of water-softening or alkali reduction, but with the theory and practice of the iron-removal processes in use in Europe and America," still there are so many waters containing iron and manganese, where these constituents form the simplest and least refractory ones to be treated, that the discussion might well include processes which will soften the water and remove, as well, other impurities far more serious than iron and manganese.

Along the Southern Atlantic and also along the Gulf Coast, well-waters containing iron are for the most part also hard. This is also true of many other well-waters throughout the country. In Western Pennsylvania, both surface and well-waters contain iron. Some of the sources of this iron are mine drainage, washings from coke ovens, and refuse from mills. Certain of these waters contain more iron than many of the supplies specifically mentioned by the author. For instance, the supply at McKeesport, Pa., has at times as much as 25 parts per million of iron, whereas the supply at Worms contains 9.5; Reading, Mass. (November, 1891, highest), 13.3; Berlin, 1.2; New Orleans, 3.2; Provincetown, Mass., 3.6.

There is no doubt whatever that for certain classes of water, including those which are naturally soft, the method described and advocated by the author will prove the cheapest and best; but a comparison of the figures derived from the diagram, Fig. 1, showing

the cost of the filtration of river waters and the deferrization of ground-waters, with the actual cost of water-softening plants which have come within the writer's observation, would indicate that water-softening processes can be installed oftentimes with very little more expense than for a plant constructed simply for the deferrization of the water; with the added advantage of giving a greater return upon the investment and producing a water which is much more satisfactory for all purposes. Where the water is hard and ferruginous, there is no difficulty about the removal of the iron by aeration or coagulation without attempting softening, and the supply is thus improved, but the relatively small additional cost of providing for the softening of the water warrants the consideration of a complete softening plant, rather than one simply for deferrization. Mr. Potter.

The water of the Youghiogheny River perhaps contains more iron than that of any other river in that part of the country, and it is present in solution as sulphates; but the amount of iron in the river was not as great as that in the well-water which, until the commencement of the operations of the softening plant, formed a part of the supply for McKeesport. The sulphate hardness, both iron and calcium, was so great at times as to warrant the abandonment of the ground-water supply altogether, especially as the geological formation in the vicinity of the water-supply station indicated possibilities of pollution from surface sources.

The presence of the salts of iron in such waters as the Youghiogheny is of great benefit in the softening process, for when acted upon either by carbonate of soda or hydrate of lime (the two constituents most generally used in water-softening processes), the hydrate of iron is precipitated. Because of its flocculent nature, this and the hydrate of aluminum, make the best coagulating agents known. On several occasions during the testing of the McKeesport softening plant, the deficiency of carbonate of soda and hydrate of lime, during the period when the iron and aluminum content in the water was high, gave rise to a colloidal condition of the iron and aluminum hydrates so that they could not be precipitated, no matter how long the period of settlement, and they passed through the mechanical filters unchanged. These iron and aluminum salts have also the well-known action of rendering the calcium carbonate (left in the water as a result of treatment) less soluble.

An added reason why water-softening should receive more general consideration in the discussion of ground-water supplies, is the value of the settling tanks in the removal of bacteria. Analyses at McKeesport show that the settling tank is performing the bulk of the work. A test made on January 25th, 1909, gave 13 000 bacteria per cu. cm. in the raw water, and in the filtered water 50 per cu. cm. Of the total amount, 99% was removed in the settling tank. On January 29th,

Mr. Potter. 1909, a test showed that with a bacterial count of 5 000 per cu. cm. in the raw water, the settling basin effluent contained 90 per cu. cm., while the filtered water was sterile. This gives an efficiency of 98.2% for the settling tank and 100% for the entire plant.

These are representative tests. The plant has been in operation for nearly four months, having been started before its full completion because of the terrible condition of the raw water in October, 1908.

MEMOIRS OF DECEASED MEMBERS.

NOTE.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

WILLIAM BEVERLY CHASE, M. Am. Soc. C. E.*

DIED OCTOBER 26TH, 1908.

William Beverly Chase, the son of Levi W. and Harriet Vining Chase, was born in Marengo, Morrow County, Ohio, on November 21st, 1852. His family was of Colonial ancestry, his great-grandfather, Beverly Chase, having served as a New York Militiaman in the War of the Revolution.

His father was an architect and master builder of the old school, and, in his early years, the son lived in the atmosphere of the builder and the artist and became acquainted with the use of the draftsman's tools.

His early education was secured in the high schools provided in the Central Ohio towns of that period, and has been described as "consisting mainly in digging out things for himself," and, to his praise be it said, that during his after life he made most successful use of the ability thus acquired.

In early manhood he removed with his parents to Southwestern Minnesota, where for five years he was engaged in surveys and in acquiring practical experience in the design and construction of railroad bridges.

In 1877 Mr. Chase removed to Oregon, and for more than thirty years was identified with the growth and development of that State. For the first three years he was occupied with map work, and was in the employ of local bridge builders, but, in 1880, his opportunity came with the construction of the Northern Pacific Railroad, surveys for which were then in progress. Beginning as a Topographer for a field party, he was soon at Headquarters in Portland preparing designs and plans for the large number of structures required for the Western Divisions of the railway, then under the charge of V. G. Bogue, M. Am. Soc. C. E.

From 1884 to 1885, he was Engineer of Bridges for the Oregon Pacific Railroad, a line crossing from the Coast through the Western portion of the State, of which the late Isaac W. Smith, M. Am. Soc. C. E., was Chief Engineer. From 1885 to 1890 he was engaged in hydraulic and sanitary surveys and work and in bridge construction for some of the towns of the State, among which were Corvallis and Eugene.

In 1891 Mr. Chase made surveys and designs for a sewer system

* Memoir prepared by D. D. Clarke, M. Am. Soc. C. E.

for East Portland, and, from 1891 to 1895, he was Engineer of Bridges for the Portland Bridge Commission, during which time he constructed the Burnside Street Bridge, crossing the Willamette River, costing $300 000, and a steam ferry-boat for North Portland. From 1894 to 1896 he was engaged in general practice, while from 1896 to 1902 he was in the service of the City, first as Superintendent of Streets and later as City Engineer, retaining the latter position until, by a change in political parties, the wing with which he had allied himself suffered defeat.

During all these years and until his last illness, he continued to act as Consulting Engineer for the County Commissioners, who under the law are charged with the duty of maintaining all the bridges and ferries crossing the Willamette within the City limits. After the retirement from the service of the City, he was engaged in making surveys and designs for various towns for water supply, street pavements, etc., the Towns of Astoria, Corvallis, McMinnville, Rainier, and Tillamook, Oregon, and North Yakima, Washington, being among the places which he served acceptably.

It was while engaged in making an examination of a water-supply project for the Town of McMinnville, in July, 1908, that Mr. Chase suffered a severe paralytic stroke from which he never recovered, being confined to his bed from that time until his death, which occurred October 26th, 1908, at the Good Samaritan Hospital, Portland.

Mr. Chase was known as a genial gentleman and a man of recognized ability and worth. He was a Christian, from early life, and although not demonstrative he was ever known as a loyal supporter of all things that make for that "righteousness which exalteth the nation." He was long connected with the Centenary Methodist Episcopal Church of Portland, Oregon, and by his loyalty and steadfastness was largely instrumental in sustaining it during a critical period of its history. His zeal in the service of his church was his by inheritance from a godly ancestry.

In 1884 Mr. Chase was married to Miss Georgia Parker of Astoria, Oregon. The death of his wife in 1894, leaving him with their family of three little daughters, and the care of his aged father, greatly modified Mr. Chase's purposes and efforts in his profession, obliging him to put aside congenial opportunities offering exercise of his energies in wider fields, but through all his life he was an inspiring example of what may be accomplished alone, following a natural bent, supplemented by faithful application and courage.

Mr. Chase is survived by two daughters, Misses Marion and Jessie Chase, of Portland, Oregon, and by a brother, the Rev. Charles E. Chase, of San Francisco, and a sister, Mrs. Lucia C. Bell, of Fruitvale, California.

Mr. Chase was elected a Member of the American Society of Civil Engineers on September 6th, 1899.

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AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

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THE SEWER SYSTEM OF SAN FRANCISCO, AND A SOLUTION OF THE STORM-WATER FLOW PROBLEM.

By C. E. Grunsky, M. Am. Soc. C. E.

To be Presented May 19th, 1909.

This paper ought to have been written and presented to the Engineering Profession long ago. It is now offered in order that suitable record may be made of the steps taken by San Francisco to better the primitive conditions which have been maintained there too long, and to call attention to a novel solution of the storm-water flow problem which is an outgrowth of the study of San Francisco conditions.

The writer was a member of the 'Board of Engineers' of 1892-93, generally referred to as the Sewerage Commission; he was one of the two 'Engineers in Charge to devise and provide a sewer system for San Francisco,' of 1893; he was in 1899 the 'Engineer in Charge,' with two associates, of the design of the sewerage system, which, with minor modifications, is now being carried out, and he was for more than 4 years, 1900-04, City Engineer of San Francisco, in which capacity he revised and re-submitted the plans of the sewer system, recast the

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

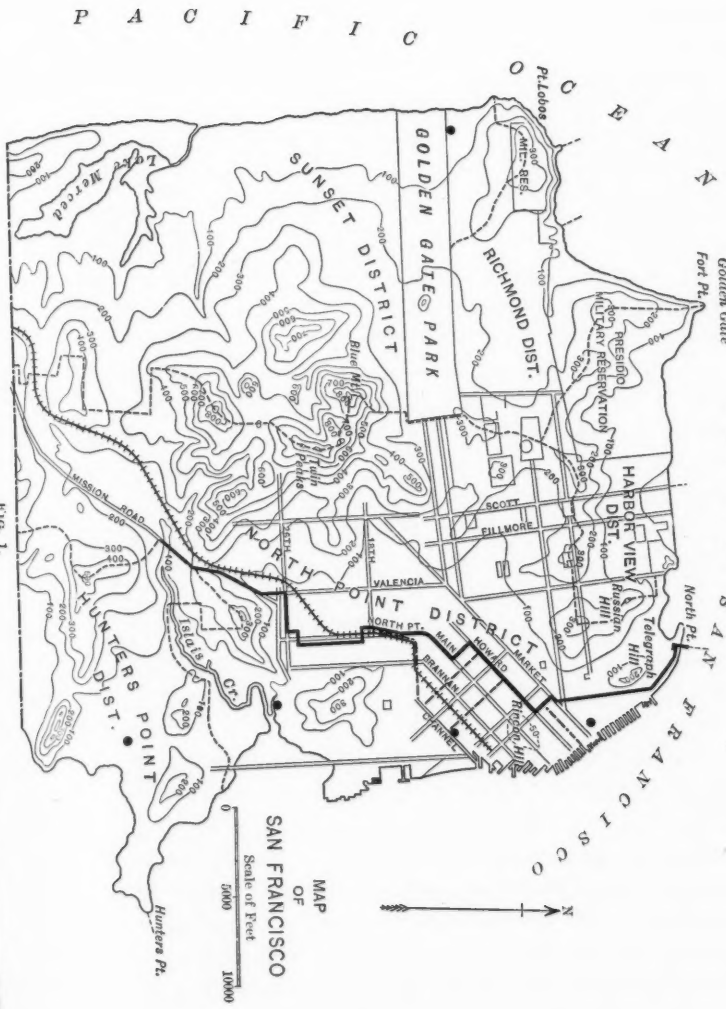
PACIFIC OCEAN
BAY OF SAN FRANCISCO
Pt. Lobos
Golden Gate
Fort Pt.
Lake Merced
SUNSET DISTRICT
GOLDEN GATE PARK
RICHMOND DIST.
MILITARY RESERVATION
PRESIDIO
HARBOR VIEW DIST.
Russian Hill
Telegraph Hill
North Pt.
Blue Mts.
Twin Peaks
MISSION ROAD
SCOTT
FILLMOREST
VALENCIA
NORTH POINT DISTRICT
NORTH PT.
MAIN
HOWARD
MARKET
BRANNAN
MARKET
Rincon Hill
HUNTERS POINT DIST.
Islais Cr.
Hunters Pt.
MAP OF SAN FRANCISCO
Scale of Feet
5000
10000
N

FIG. 1.

cost estimate thereof, and, as funds permitted, directed the construction of the first elements of the new system. This statement is offered in explanation of the free use which has been made in this paper of the reports on the sewerage problem of San Francisco, prepared or participated in by the writer from time to time.

Topographic Features.

San Francisco covers the entire northerly portion of a peninsula in which a spur of the Coast Range, coming up from the south between San Francisco Bay and the Pacific, terminates. The city is irregularly square in outline. On the west is the Pacific Ocean; on the north, the ocean, the Golden Gate, and the bay; on the east, the bay; and on the south, Marin County, from which San Francisco is separated by an east and west line extending from the bay to the ocean.

The area of the city is about 30 000 acres, of which about one-third is built-up territory. The built-up section covers the northeastern quadrant of the city, and extends, in long narrow strips, westward, in the northern half of the city, to the ocean, and southerly along such principal lines of travel as the San Bruno and Mission Roads to the County line. Within recent years the growth of the section on the ocean slope, particularly the region just south of Golden Gate Park, has been rapid.

The highest point in the city is practically in the city's geographical center. Here are the Twin Peaks which rise to an altitude of a little more than 900 ft. A short distance to the northwest of Twin Peaks is Blue Mountain, with a summit at practically the same height.

The ridge which divides the city into two slopes, one easterly toward the bay, the other westerly toward the ocean, enters the city from the south, a little west of a median line, and holds a fairly direct northerly course to a termination at Fort Point. The lowest points on this ridge are more than 200 ft. high. Before the recent completion of the bay shore cut-off, by the Southern Pacific Company, its main line of railroad into the city crossed this ridge from east to west just south of San Francisco, and within the city crossed back over it from west to east to reach the east central portion of the city. Parts of Golden Gate Park and the Richmond District lie upon the flat top of this ridge northward from the Twin Peaks group of hills.

The principal spurs within the city, from this peninsula backbone, which subdivide the eastern or bay slope, are, in their order from south to north: Hunter's Point ridge, terminating in a cape which extends far out into the waters of the bay in the southeastern portion of the city (south of this spur are two small valleys opening to the bay, of which the southernmost, Visitacion Valley, lies in part in San Mateo County); Bernal Heights, between two branches of Islais Creek; the Potrero Hills, which are barely connected with the main ridge by a slight swell in the ground; and the northern ridge, terminating in Russian and Telegraph Hills on the northeastern bay front of the city.

The western water-shed or ocean slope is subdivided, by the flat-topped Ocean House ridge and the Point Lobos spur, into three drainage basins, of which the northernmost is drained by Lobos Creek; the second embraces a region of drifting sand dunes extending well up on the western slope of the central group of hills; and the third is the northern part of the water-shed of Merced Lake.

Some of the hills near Point Lobos are more than 300 ft. high; the Fort Point ridge, within the Presidio, has a height of more than 400 ft.; the Presidio Heights, near where the northern ridge leaves the main ridge, have greatest elevations of 300 to 380 ft.; Lafayette Square rises to a height of 380 ft.; Clay Street Hill is 360 ft. high; Russian Hill, 370 ft., and Telegraph Hill, more than 290 ft. The lowest point on the spur which extends northwesterly from Russian Hill and terminates at Black Point is about 90 ft. high. On the Potrero Hills are points nearly 340 ft. high, and on the Hunter's Point ridge several hills rise to more than 250 ft. Railroad Avenue crosses this ridge at an elevation of 60 ft., and San Bruno Road at the same elevation. The spur which extends eastward from the main ridge south of the Hunter's Point ridge near the south line of San Francisco, leaves the main ridge with an elevation of 550 ft. This ridge separates Bay View Valley from Visitacion Valley. It is crossed by the San Bruno Road at an elevation of 140 ft., and terminates with a hill the top of which is 400 ft. high.

Bernal Heights rise to more than 490 ft.; Buena Vista Park is nearly 520, and Lone Mountain nearly 480 ft. high.

Between the several ridges or spurs of hills just mentioned lie valleys or broad flat slopes opening out upon the bay or ocean. The

ocean slope water-shed south of the Point Lobos ridge has no well-defined line of drainage. Irregular sand dunes, formed of sand blown eastward from the ocean beach, form a broad broken slope to the ocean. Some of the drifting sand has been carried over the main peninsula ridge just north of the central group of hills, and, separating into two parts, at Lone Mountain, has found lodgment in several broad belts extending eastward and southeastward across the peninsula to Mission Bay. In smaller quantity, some sand has also been carried across the peninsula south of Twin Peaks.

Sand has also been blown inland from the bay beach lying on the northern front of the city to the westward of Black Point. The great sand dunes, now only seen in small remnants, that were formerly at and near Lobos Park and some distance up on the slopes to the east and north, had their origin on this beach.

Where sand has thus drifted over portions of the site of the city it covers clays, shales, and serpentines. To some extent it encroached upon the waters of the bay on the eastern waterfront adding body to the marginal strips of swamp land and mud. Where the natural surface of the city is not covered by sand a clay soil predominates. On the hills a shaly or serpentine rock is frequently cut into when trenching for sewers, gas, or water pipes.

The northern and eastern shore of the city in its original condition was deeply indented by arms of the bay. These have in part been filled in. The most notable of these filled-in areas is the former Yerba Buena Bay, which gave the original name to San Francisco. This extended as far west as Montgomery Street and from the base of Telegraph Hill at the north to the Rincon Hill region on the south. Mission Bay, too, has been cut off and filled in. The extent of this filling will be appreciated when it is stated that here, over an area of about 150 acres, all official street grades are at City Base (6.7 ft. above ordinary high tide).

Islais Creek, south of the Potrero Hills, opens to the bay through broad marsh lands and mud flats, 500 acres of which are within the city limits. It is in this region that, fully two miles inland, official street grades as low as 1 ft. below City Base may be noted.

South of Hunter's Point an area of from 700 to 800 acres within the city limits extends out into the bay over submerged mud flats which will probably some time be filled in to about City Base.

On the northern front of the city the tidal area and bay surface that have been or that ultimately will be cut off from the bay by the established seawall line aggregate about 270 acres.

The total area of originally salt-marsh lands and bay surface within the boundaries of San Francisco is nearly 2 000 acres.

The built-up or improved sections of San Francisco fall naturally into districts of two characters: one class embracing portions of the city on sloping ground, hillsides, and hill-tops, at such elevations that they can be readily drained by gravity flow; the other embracing the low flat areas for which it is difficult or impossible to provide satisfactory sewage collection without recourse to pumping. The several districts of each class, as the foregoing description of the topography indicates, do not lie contiguous to each other. The drainage and sewerage problem is thus rendered much more complex than if there were only one instead of many drainage basins.

Water-shed Areas.—The extent of the several water-sheds, the run-off waters of which are to be cared for was in 1893 estimated as follows:

Visitacion Valley water-shed	990 acres.
Bay View water-shed	1 220 "
*Islais Creek water-shed	3 900 "
Precita Creek water-shed	1 730 "
Mission Bay water-shed	5 860 "
Yerba Buena Bay water-shed	430 "
North Beach water-shed	320 "
Washerwoman's Bay water-shed	1 030 "
Presidio water-sheds	930 "
Lobos Creek water-shed	2 120 "
Ocean Beach water-sheds	5 650 "
*Laguna de la Merced water-shed	3 350 "

In this list Precita Creek has been given independent rank. It is a tributary of Islais Creek, uniting with the latter at the margin of the salt marsh. Bernal Heights forms a prominent line of division between these two creeks.

Visitacion and Bay View Valleys.—Both Visitacion and Bay View Valleys are still sparsely settled, but the latter at least is a section in immediate need of main sewers.

* Only the area of these water-sheds within San Francisco is here noted.

Islais Creek.—The main branch of Islais Creek heads to the southward of San Francisco. It descends rapidly in its westerly course to the low marsh lands which extend far up into the valley of this creek. The creek before emerging upon these marsh lands lies for some distance in a deep gorge which takes a course through private lands. The rectangular system of laying out streets has been adhered to in some places along this creek, where, without question, there should have been streets following lines of drainage. Where lands are not yet subdivided, it is to be hoped that this matter will receive proper attention.

Islais Creek receives the run-off waters from a portion of the San Bruno Mountain, to the south of San Francisco, from the northeast slope of the Twin Peaks group of hills, and from the entire southern slope of Bernal Heights. The region which it drains, the south central portion of San Francisco, is a rapidly growing section of the city.

By statutory enactment, Islais Creek is declared to be navigable "from Franconia Landing near Bay View turnpike to its outlet into the Bay of San Francisco, and thence easterly along the southerly line of Tulare Street to the city water front on Massachusetts Street, of the width of the channel of said creek." However, Kentucky Street has been carried across the creek on a solid fill and there is still some doubt as to whether the creek, which across the marsh is but a slight depression in the mud's surface, will be opened and kept open for navigation purposes. But at the mouth of the creek a large basin should be dredged to a depth of about 40 ft. to meet in part the needs of San Francisco's growing commerce. It is foreseen that a project for such a basin will ultimately be carried out. All the Islais Creek marsh has been laid out in streets, and official grades have been established. Sometimes this was done in the most absurd fashion. Thus, on San Bruno Road and on Fifteenth Avenue, grades are as low and even lower than on Kentucky Avenue, 1¼ miles nearer the bay.

Precita Creek.—This creek drains a region which lies between the Islais Creek water-shed on the south and the Mission Bay drainage on the north. As a drainway, the creek has been replaced by Army Street sewer. The discharge of sewage is upon or through the marsh into Islais Creek, a short distance east of the eastern spur of Bernal Heights.

Mission Bay Water-shed.—The drainage basin tributary to Mission Bay is of large extent, and is for the most part densely populated. It includes the heart of the city, the entire Mission and the Western Addition. All the sewage and storm-water run-off of this region, except a relatively small portion intercepted by Brannan Street sewer, finds its way into the open waterway of Channel Street. The large Channel Street sewer, and the sewers on Seventh, Sixth, and other streets, discharge into this waterway. The discharge from these sewers represents more than half of the sewage and rain-water of the built-up portion of the city. During half the year there is no rain to dilute the sewage, and it ebbs and flows in Channel Street, creating a filthy nuisance beggaring description. There is no chance for sweeping it out into the bay because there is no circulation of water. It will continue to be offensive until the sewage is carried to some other point of outfall. But even under the contemplated arrangement to be hereinafter described the remedy will not be complete. There will still be some, though very dilute, sewage occasionally discharged into Channel Street, and this open water will remain under a disadvantage common to all long narrow inlets from a tidal bay, that of infrequent change of its water body.

The principal sewer of the Mission Bay water-shed, as now in service, is the Channel Street sewer, 11 ft. wide and 8½ ft. from invert to roof. Second in importance, if judged by size, is the Brannan Street sewer 9 ft. wide and 7 ft. high in the clear. It intersects the sewers of crossing streets; and, by reason of a connection at its head, near Tenth Street, with the Channel Street sewer, is supposed to act in a measure as a relief outlet for it. The grade of the Brannan Street sewer does not conform closely to the grades of the sewers which it intersects. These intersected sewers are generally from 1 to 3 ft. higher than the bottom of the Brannan Street sewer, which thus should become a recipient of the sewage from the entering sewers. As a matter of fact, it is found that the Brannan Street sewer was constructed so carelessly that, where in tunnel, under the south slope of Rincon Hill, there is an offset of 10 or 15 ft., and its bottom grade for several blocks slopes the wrong way. The accumulation of silt in the sewer obliterates the non-conformity of grade with the intersecting sewers and there is therefore a partition of flow at practically every such sewer crossing.

The Brannan Street sewer discharges into the bay under the First Street wharf.

Other sewers on Mission, Howard, and Folsom Streets, have independent outfalls, under the wharves or piers of the waterfront.

The sewage from the northern slope of the Potrero Hills is discharged by small sewers upon the Mission Bay mud flats or, reaching the Potrero Street sewer, it is delivered into the bay in the Potrero near the Union Iron Works.

Yerba Buena Bay Water-shed.—The name, Yerba Buena, as applied to any section of San Francisco is now practically lost. Yerba Buena Bay, the old harbor of Yerba Buena, the precursor of San Francisco, has long ago been filled in and built upon. The extreme westerly end of the bay, that is to say, the point of greatest indentation, reached Montgomery Street. The name has been used in the various sewer reports to designate the low, flat portion of San Francisco which lies north of Market Street, near the bay, and the water-shed includes the slopes of the hills to the north and the west which send their drainage through this flat area. As a sewer district, though of small extent, it is of particular interest and importance, first, because about 120 acres thereof are of the filled-in character, second, because it embraces the most important business section of the city. In this old part of San Francisco are to be found the most striking examples of sewer construction, without comprehensive plan. Almost every street and alley has its 3 by 5-ft. sewer (see Fig. 2), and at every street crossing there is the typical sewer intersection. North and south streets with level surfaces, that is, without gradient, are no exception to the rule. The street in which the standard brick sewer cannot be found is a welcome exception. Until about 1890 there was an outlet for each east and west sewer at the water front. Then, however, a number of these sewers were diverted on East Street from their direct courses and a common outfall at Washington Street was provided for Market Street sewer and the sewers as far north as Jackson Street. This interception of the sewage from the west, which includes the intersection by the lower Market Street sewer of all flow in the sewers from Geary Street to Sacramento Street, makes the Washington Street outlet a point at which a large quantity of sewage is discharged. This outlet is at the low-water line, under the wharf.

The sewers of Vallejo and Pacific Streets have been turned into the

Broadway sewer, which thus becomes their common outlet. It, also, discharges under a wharf at the low-water line. It follows, from the conditions indicated by this description, that much slush and filth has found lodgment in the sewers of this flat area; and it may be added that the gradual subsidence of a part of the area, and the construction of sewers, a block or two at a time, which did not conform to the other sewers in grade, sometimes being several feet too high or too low, have greatly augmented the evil.

FIG. 2.

The conditions, as they were found when the first examination of this district was made, in 1892, plainly pointed to the use of the existing sewers, in part, at least, as storm-water conduits, and the construction of a new system for sewage only.

North Beach Water-shed.—The North Beach region has a long frontage on the bay. It extends from Telegraph Hill to Black Point.

The main difficulty to be overcome in this region results from the long bay frontage. Sewers now in use discharge into the bay wherever they happen to reach the water line. Sewers of this character are those on Larkin, Hyde, Mason, and Powell Streets.

Washerwoman's Bay Water-shed.—From the Presidio Heights eastward to Russian Hill and Black Point, the northern slope of the northern ridge is toward that part of San Francisco Bay known originally as Washerwoman's Bay. Here, again, the peculiarity of a steep descent from the ridge and the large extent of comparatively flat marginal land is to be noted. Some of the land along the bay remains to be reclaimed.

The built-up area is here fast extending downward on the hillsides toward the bay. Sewers for a long series of years terminated in land-locked basins or at the inshore limit of the tidal marsh. Some of the principal sewers have more recently been extended to the bay shore. The present outfall points of this district are at the bay shore on Pierce Street, on Steiner Street, on Fillmore Street, under a wharf, and on Webster Street.

Lobos Creek.—Lobos Creek had its head originally in Mountain Lake. Though not fed by any well-defined natural lines of surface drainage, it includes within its water-shed all the Richmond District. The southeastern limit of its water-shed is at Lone Mountain. The plateau region northward from the Lone Mountain ridge, westward from First Avenue, and southwestward from the Presidio ridge is drained by this creek. For convenience of classification, all the ocean front drainage, from Fort Point to Point Lobos, may be included in the Lobos Creek water-shed.

The principal sewer of this district is the Richmond main. Here, at variance with the rest of San Francisco, an orderly arrangement of the sewers, which are of comparatively recent construction, may be noted. The outfall of the Richmond main is just west of Baker's Beach, at the foot of Twenty-seventh Avenue.

The Ocean Beach Water-shed.—This region fronts on the ocean, and extends from Point Lobos to Laguna de la Merced. It is without natural lines of drainage, except the small gullies on the western slope of the Twin Peaks group of hills which are soon lost in the deep irregular sand dunes which have drifted in from the ocean.

Laguna de la Merced Water-shed.—Lake Merced lies in the south-

western portion of the city, but a few feet above sea level. The lake has a surface area of about 300 acres. Of its total water-shed of 7 500 acres, somewhat more than one-half lies south of the city in San Mateo County.

The lake is still in use as a source of water for San Francisco, being fed by the waters which percolate through the sands covering a large portion of its water-shed. Some 2 700 acres, of which about 2 300 lie within San Francisco, are owned by the Spring Valley Water Company, around the lake, for its protection against pollution.

The surface run-off from the south is intercepted and turned through a tunnel into the ocean. Likewise, the storm-waters of the Ocean View region are brought within reach of the same tunnel by a long flume. The sewage of Ocean View is carried in a line of cast-iron pipe to an outfall into Merced Creek below the lake.

Studies for a Sewer System and its Design.

The need for a comprehensive system of sewers in San Francisco was pointed out, with increasing frequency and emphasis, by various city officials and committees, until in 1892 the Board of Supervisors, after conference with Frank Soulé, M. Am. Soc. C. E., of the University of California, passed the following resolution, dated March 7th, 1892:

"*Resolved*, That Professor George Davidson of the United States Coast and Geodetic Survey, Col. George H. Mendell of the United States Engineers, and Irving M. Scott, Esq., be and are hereby empowered to select and appoint two engineers, and when the same are so appointed that they, with the gentlemen named, shall constitute a Board of Engineers on and from September 1st, 1892, to devise and provide a system of sewerage for this city and county, as proposed in the report of the Committee on Streets, etc., of this Board, made January 25th, 1892."

At a meeting on August 8th, 1892, the first three members of the Board of Engineers then created:

"*Resolved*, That Marsden Manson* and C. E. Grunsky be appointed as the two civil engineers to complete the Board of Engineers, as contemplated by Resolution No. 6612, adopted by the Board of Supervisors, March 7th, 1892; provided, they agree to devote their entire time to this work after being officially notified by Professor Davidson."

It quickly became apparent to the Board of Engineers thus created, that there were no adequate records of the sewers in use. The sewers

* Member, Am. Soc. C. E., and now City Engineer of San Francisco.

had for the most part been constructed piecemeal, a block or a street crossing at a time, and diagrams to illustrate the work had frequently been considered superfluous. In many cases no record had been made of the completed structures. It was necessary, therefore, to make an examination of the sewers by entering manholes and by exploring the larger sewers. In this way dimensions and conditions as to serviceability, if retained in use as a part of a comprehensive system, were ascertained. Several exploring parties were organized for this purpose, and the results of their work were at once mapped on a suitable scale.

The preparation of a complete topographic map was also undertaken and this was subsequently found to be of great value in facilitating the studies.

Attention was at once given by the Board to the selection of suitable points of outfall for the city's sewage. It was assumed at the outset that only one method of disposal—discharge into the bay or ocean—would come under serious consideration. Therefore, plans were made to observe the direction and velocity of the bay currents.

The work of the Board of Engineers had not been far advanced before its independent course gave offense in certain quarters and led to its dismissal (January, 1893). Its books and the data collected in the four months of activity were by the Board of Supervisors turned over to Mr. Marsden Manson who, however, declined an offer to continue the work. A few days later, after repeated conferences with members of the Board of Supervisors, Mr. Manson and the writer were appointed as engineers in charge to devise and provide the sewer system.

The work outlined by the Board of Engineers was continued, and has become the basis for all sewer work in San Francisco since that time. On July 1st, 1893, the existing sewers, 200 miles in length, had been examined and their general condition noted. All sewers through which it had been possible for a man to pass had been examined as to general condition. In some instances, where data could not be obtained otherwise, the sewers were uncovered. The ascertained facts were tabulated for convenient reference, and were platted on sheets of convenient size on a scale of 100 ft. to the inch. Sixteen of these sheets were at that time complete, ten were in progress, and forty more would be required to cover the built-up areas of the city. At this time, the general map of the city, on a scale of 600 ft. to the inch, had been nearly completed.

Float observations at all stages of the tide were made in the bay from a number of points, principally Hunter's Point, Potrero Point, Center Street Pier, and Powell Street Pier. The purpose of these observations was to make sure that the discharge of sewage at the outfall points, as finally selected, would, by reason of ample dilution, become inoffensive and harmless before any of it drifted inshore.

The work of the Engineers in Charge, however, was not carried forward to the completion of a design of a system of sewers. They shared the same fate as the Board of Engineers—summary dismissal in the fall of 1893—and nothing further was done until 1899 when the Board of Supervisors determined to continue the work under the same engineers but with the addition of the City and County Surveyor, the late Charles S. Tilton; this addition being made as a matter of policy. The resolution authorizing the continuance of the work was passed in May, but, owing to some uncertainty regarding its legality, it had to be re-enacted in modified form. This took time, and it was not until late in September that it seemed certain that the funds set apart for the work would become available. In the meanwhile the work was under way and was rapidly pushed to completion before the close of the year.

Before submission to the Board of Supervisors the report of 1899 on the sewer system was passed upon by Rudolph Hering, M. Am. Soc. C. E., and received his endorsement.

Some Features of the Problem.

The question of determining whether sewage and rain-water should be carried in the same or separate conduits was necessarily one to be determined for each of the many districts into which the city is divided by its topographic features. The method and place of disposal and the sewers already in service were elements requiring to be weighed in this determination. It was also necessary to make a careful study of rainfall records, in order to ascertain the volumes of storm-water flow that would have to be passed through the sewers to the bay or ocean.

Before taking up the discussion of these matters some additional notes relating to the old sewers, to show the general lack of system in their arrangement, may not be out of place.

The statement, that the plan for providing sewers seems to have been to construct egg-shaped brick sewers, 5 ft. high and 3 ft. wide, in

all streets and alleys, where property was valuable and could afford to pay for large sewers, as applying to most of the sewer work that had been done in San Francisco, is substantially true. The size of sewer required was frequently determined by the Superintendent of Streets, who was never a civil engineer, and the prescribed depth of the sewer, as defined by ordinance, was 10 ft. below the official grade of the street. Therefore, sewers are found, as already explained, where they are all but useless, and types of construction have resulted that are certainly unique. These facts were set forth at some length in the Progress Report of 1893 of the 'Engineers in Charge.'

For years the ordinary improvement of street crossings included sewers across the streets in both directions and storm-water inlets at the curbs on the four corners. The sewers in such intersections were built of brick—four wings from a central manhole. The invert, as required by ordinance, was placed 10 ft. below street grade, generally level, or, due to the intelligence of most of the sewer contractors, a few inches low at the down-hill side of the street intersection. The sewers in the intersection might connect with other brick sewers of like size, or with larger sewers or with small pipe sewers, according to what was prescribed at some other time for the streets leading from the intersection. After a time, there was a modification of this arrangement of sewers at intersections of streets. Where it was known that the property owners of outlying districts could not afford the large brick sewers, pipe sewers were prescribed for the crossings. The "Bobtail" crossing was introduced. This consists of a section of 5 by 3-ft. sewer 16 ft. long placed along the axis of one street with pipe sewers from its ends and with pipe sewers along the axis of the cross street. A final modification for crossings of small sewers consisted in omitting altogether the brick sewer and putting in two practically horizontal pipe sewers along the center lines of both intersecting streets.

There is no excuse for the many useless storm-water inlets that have been constructed on gutter summits. These occur in all parts of the city which had been improved to any extent before 1890. In 1893, in a central portion of the residence district of the city, a count was made of the storm-water inlets located at corners from which there was a down gradient in both directions, into which, therefore, no water can flow. The district covered by the count extended from Taylor Street on the east to Broderick Street on the west, and from Washington

Street on the north to Hayes Street on the south. In it there are 1 080 storm-water inlets, of which 149 are absolutely useless, and many more receive water from so small an area of street surface that they are to be classed as unnecessary. Each of these useless and unnecessary storm-water inlets, or catch-basins, as usually called, cost the property owners about $90.

In San Francisco, as in most of our cities, the improvement of streets and the construction of local sewers is paid for by the property which fronts on the improvement. The work may be done in districts, as was done in the Richmond District, when the main lines of drainage were put in, and in the case of the Army Street sewer; but, ordinarily, only short stretches of sewer were covered by the official order of construction or in advance of such order, under which there would be a public letting of the work to the lowest bidder, the property owners would be prevailed upon to enter into a private contract to do the work. The favored contractor was at times responsible for the size of the sewer, suggesting it and securing official approval; at other times the Superintendent of Streets and Sewers determined the character and size, at any rate, this matter was subject to his approval. Only in the minority of cases was it possible to refer to a general plan covering some section of the city for sewer sizes.

The city, preceding 1900, had a City and County Surveyor, whose principal duties related to lot surveying. This was an elective office. As the law also required that a City Engineer be appointed by the Board of Supervisors, it was customary for the Board of Supervisors to appoint the incumbent of the elective office to the office of City Engineer. For services rendered in the capacity of Surveyor or Engineer fees were collected, and when special work of any kind was required by resolution or order of the Board of Supervisors, the compensation to be made was usually named therein.

Supervision of construction was in the hands of the Superintendent of Streets. The City Engineer staked out line and grade and issued a certificate, showing satisfactory construction or showing by diagram departures from prescribed alignment or grade. He was not held responsible for the acceptance of work, however great the deviation from a required position might be.

This unsatisfactory method of prescribing and carrying out street and sewer work continued until the present city charter went into

effect, in January, 1900. Since that time a Board of Public Works has had charge of street and sewer construction, and under a well-organized Bureau of Engineering, with a salaried City Engineer at its head, all matters relating to sewer design and construction receive competent and proper attention.

A few diagrams were prepared in 1893 by the Engineers in Charge, to illustrate the lack of system, and peculiarities of the treatment of sewerage problems in San Francisco. Of these only one, Fig. 2, showing the almost universal use of 3 by 5 ft. brick sewers, is exhibited herewith.

Rainfall in San Francisco.

When the studies relating to the design of a comprehensive sewer system for San Francisco were commenced in 1892, an inquiry was at once made of the Weather Bureau for data relating to rates of precipitation. All that could then be learned was that there were no rains of an inch or more per hour, and that therefore the Weather Bureau had not undertaken to record the rate of rain. But it was agreed that the matter should at once be taken up, as it was quite as important for San Francisco to have positive evidence that the maximum rain in an hour was about 0.5 in. as for the cities of the Atlantic Slope to know that 2 in. or more per hour may be expected occasionally.

Therefore, in 1899, when the matter was taken up again, enough information was at command to enable satisfactory conclusions relating to maximum rain intensities to be reached. Mr. Thomas Tennent, a retired maker of nautical instruments, who had for nearly 50 years kept a rainfall record, was of assistance in the rainfall study. His records were placed at the writer's disposal* and were freely used. From them and from those of the Signal Service and United States Weather Bureau (commencing March 9th, 1871), it was found that there had been rain in excess of 2 in. per day as shown in Table 1.

In order that the infrequency of the days on which there is rain in excess of 2 in., and their occurrence, unless as a rare exception, only in the months November to April, may be understood, some further notes on rainfall and climate may not be out of place. Very little rain is expected from May 1st to November 1st. There are no violent

* At the writer's suggestion, these records were turned over by Mr. Tennent to the U.S. Weather Bureau. They were lost in the fire of 1906.

thunderstorms, as in the States of the Atlantic Slope, the Middle West and the Central Northwest. Even in the winter, rain storms accompanied by thunder and lightning are of rare occurrence.

TABLE 1.—RAINFALLS, AT SAN FRANCISCO, IN EXCESS OF 2 INCHES PER DAY.

Date.	Inches.	Date.	Inches.	Date.	Inches.	Date.	Inches.
Jan. 29, 1850....	2.20	Jan. 9, 1862....	3.50	Feb. 20, 1867....	2.12*	Dec. 23, 1884...	2.01
Dec. 17, 1852....	3.00	Jan. 10, 1862....	2.46	Feb. 21, 1867....	2.22	Nov. 23, 1885...	2.58
Dec. 25, 1852....	2.54	Jan 16, 1862....	2.46	Dec. 18, 1871....	2.83	Dec. 21, 1885...	2.78*
Jan. 7, 1853....	2.06	Jan. 17, 1862....	2.64	Dec 19, 1871....	3.12	Jan 23, 1886...	2.35
Mar. 27, 1853....	2.85	Feb. 21, 1862....	2.09	Dec. 23, 1871....	2.48	Feb. 4. 1887...	2.22
Apr. 16, 1853....	3.45	Nov. 26, 1864....	3.98	Jan. 8, 1872....	2.36	Feb. 5. 1887...	2.92
Mar. 13, 1854....	2.25	Dec. 12, 1864....	2.56	Nov. 29, 1872....	2.06	Mar. 13, 1889...	2.54
Oct. 21, 1858....	2.06	Jan. 17, 1866....	2.17	Nov. 23, 1874....	3.98	Feb. 15, 1891...	3.38
Mar. 26, 1861....	2.53	Jan. 20, 1866....	2.22	Nov. 17, 1875....	2.30	Dec. 29, 1891...	2.21
Dec. 26, 1861....	2.02	Dec. 19, 1866....	4.28	Mar. 5, 1879...	2.73	Nov. 23, 1896...	2.00
Jan. 5, 1862....	2.67	Dec. 20, 1866....	3.62	Jan. 29, 1881....	4.67	Mar. 22, 1899...	2.15

* In 11 hours.

The cyclonic disturbances that pass over the country from west to east—usually from two to four per month—are generally of great extent. When they are accompanied by rain, there is ordinarily a steady downpour at a moderate rate until the area of low barometer has passed on to the eastward, when the change of wind direction from southeast to southwest and the fall in temperature may be accompanied by heavy clearing-up showers. These showers, which are not frequent, are generally local. They may cover only a small portion of the area of San Francisco, and generally have sharply defined limits. Even when the entire city is shower swept, the heavy fall of rain is not simultaneous in all parts of the city. Neither are the maximum rates of rain long sustained. The fact that the maximum rainfall in one hour is well below 1 in. is clearly established by all the records of rain that have been kept in San Francisco. On this point, under date of June 29th, 1899, Mr. Thomas Tennent says:

"In reply to your inquiry concerning the rainfall in San Francisco, and what has been the greatest quantity which has fallen in a given time, say one hour, I will state that I have personally kept a record of the rainfall in this city from 1849 to 1896, measuring the quantity in each 24 hours at 8 A. M. On looking over my records [preceding the establishment of the U. S. Signal Service in March, 1871] I find that the heaviest rainfalls we have had in 24 hours occurred, viz.: 1864,

November 26th, 3.98 in., with the remark, 'Heavy rains in the morning; rain and gale at midday; steady rain P. M. and evening.' 1866, December 19th, 4.28 in., 'heavy rain and gale in the morning; hail with thunder at midday, rain during the balance of the day and evening.' 1866, December 20th, 3.62 in., 'high wind and cloudy in the morning; then a steady rain all afternoon and evening.' I can state without hesitation, that we have never had a fall of rain of 1 in. in any one hour."

Based on the records available in 1899 it was found that:

The average number of rainy days per year is 66. There are to be expected no days on which it rains 5 in., and the probabilities are:

1	day	in	50	years	with	more	than	.....	4.50 in.
1	"	"	25	"	"	"	"	.....	4.00 "
1	"	"	10	"	"	"	"	.....	3.50 "
1	"	"	6	"	"	"	"	.....	3.00 "
2	days	"	5	"	"	"	"	.....	2.50 "
1	day	per	year		"	"	"	.....	2.00 "
2	days	"	"	"	"	"	"	.....	1.50 "
6	"	"	"	"	"	"	"	.....	1.00 "
9	"	"	"	"	"	"	"	.....	0.75 "
16	"	"	"	"	"	"	"	.....	0.50 "
26	"	"	"	"	"	"	"	.....	0.25 "
40	"	"	"	"	"	less	than	.....	0.25 "

The frequency of rain and the number of days on which rain fell in excess of the several amounts above named is shown by the diagram, Fig. 3, as determined by actual count for a period of 50 years, from 1849 to 1899. The close agreement between the fact and the probability, indicated by the curve, is noteworthy.

From this study it was concluded that rain storms with excessive fall of rain are rare; that nearly two-thirds of the rainy days are days with less than ¼ in. of rain; and that any day with rain in excess of 5 in. is highly improbable.

It should be stated, too, that on the days that show light rain the precipitation is usually well distributed, the rain is light and continuous during the greater part of the 24 hours, more frequently than in the form of showers of short duration.

The data secured by the U. S. Weather Bureau subsequent to the inquiry of the Board of Engineers in 1893, relating to rain intensity,

were carefully studied. As already stated, there had been a sufficient number of heavy rains to enable satisfactory conclusions to be reached.

On	January 20th, 1894,	it	rained	0.15	in.	in	5	min.
"	October 12th, 1899,	"	"	0.09	"	"	"	"
"	November 23d, 1896,	"	"	0.08	"	"	"	"
"	January 20th, 1894,	"	"	0.17	"	"	10	"
"	November 23d, 1896,	"	"	0.14	"	"	"	"
"	November 23d, 1896,	"	"	0.55	"	"	1	hour
"	January 20th, 1894,	"	"	0.36	"	"	"	"

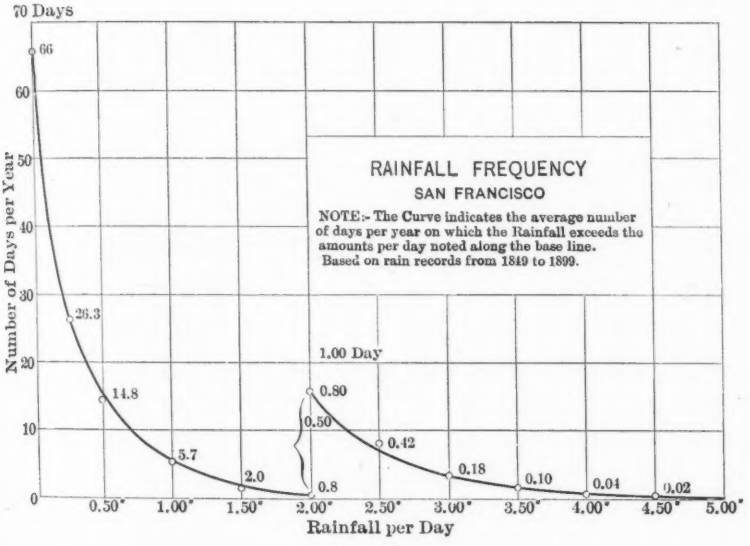

FIG. 3.

Using these and other data from the records, a curve of rainfall intensity was prepared, or rather a limiting curve to show the maximum amount of rain to be expected in any time unit up to 24 hours. The lower portion of this curve was platted on an enlarged scale. This curve is shown by Figs. 4 and 5.

Based on rainfall measurements made subsequent to 1899, a new curve has come into use, which shows somewhat greater rain intensities for short time-periods. The curve now in use is shown in dotted lines.

The most important fact relating to rain intensity disclosed by

the U. S. Weather Bureau records since 1899* is that there was one rain storm in the spring of 1904 which exceeded the intensities indicated by the curve now in use, by about 10% for all periods of time up to one hour. Only two other rain storms were of greater intensity than those indicated by the curve, but the excess in one case was only for a 5-min. period, in the other for a 10-min. period. It is believed that the heavy rain in the spring of 1904 did not cover the entire city.

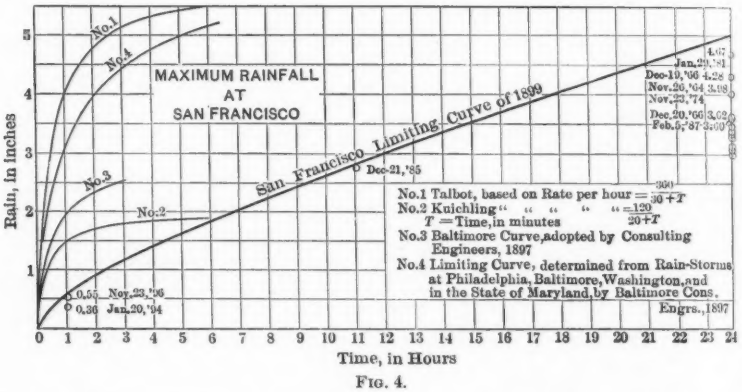

FIG. 4.

The interpretation of the diagram prepared in 1899 established the following for San Francisco:

The maximum amount of rain to be expected:

In	any	5-min.	period	is	0.15	in.
"	"	10	"	"	"	0.22 "
"	"	20	"	"	"	0.32 "
"	"	30	"	"	"	0.40 "
"	"	40	"	"	"	0.47 "
"	"	50	"	"	"	0.54 "
"	"	60	"	"	"	0.60 "
"	"	2-hour	"	"	"	0.90 "
"	"	3	"	"	"	1.15 "
"	"	6	"	"	"	1.83 "
"	"	12	"	"	"	3.00 "
"	"	24	"	"	"	5.00 "

* From facts furnished by H. De H. Connick, Assoc. M. Am. Soc. C. E., Assistant City Engineer.

It was confidently assumed that these quantities of rain would rarely be exceeded, and, if exceeded, that the area covered by the excessive rain would probably be small; that therefore these quantities might safely be accepted in estimating maximum storm-water run-off.

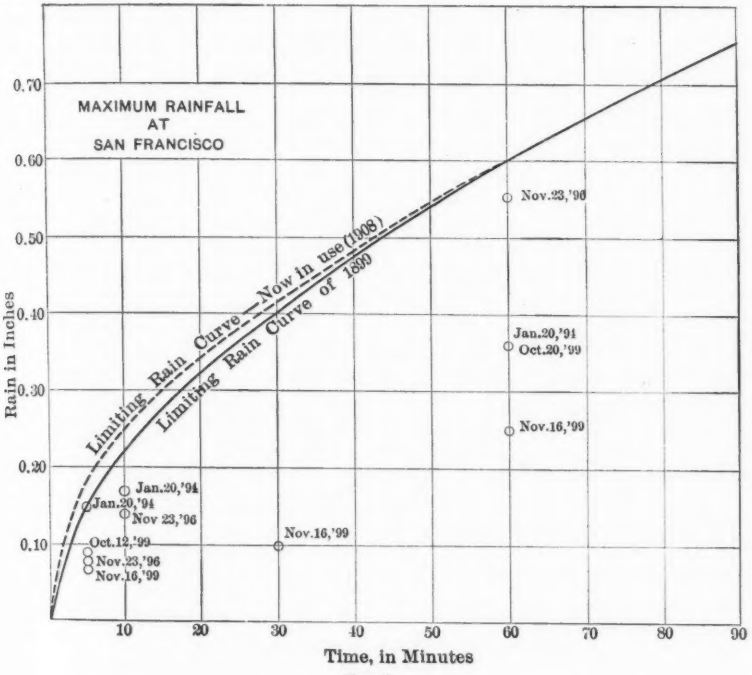

FIG. 5.

Bearing in mind the fact that a rain expressed in inches per hour is practically equivalent to the same number of cubic feet per second per acre, it follows from the above that it may rain in San Francisco as follows:

Duration, in minutes.	Rate, in inches per hour.	Equivalent, in cubic feet per second per acre.
5	1.80	1.80
10	1.32	1.32
20	0.96	0.96
30	0.80	0.80
40	0.70	0.70
50	0.65	0.65
60	0.60	0.60

This is represented graphically in Fig. 6. The ratios above noted were used in estimating the required capacity of storm-water sewers. They have been modified somewhat since 1899, as stated, for the shorter time periods, so that at present, when a sewer for a small district is designed, somewhat larger values for storm-water flow are obtained.

For comparison, the limiting rain curve now in use is shown as already stated in Fig. 5, and the corresponding maximum rainfall rates in Fig. 6.

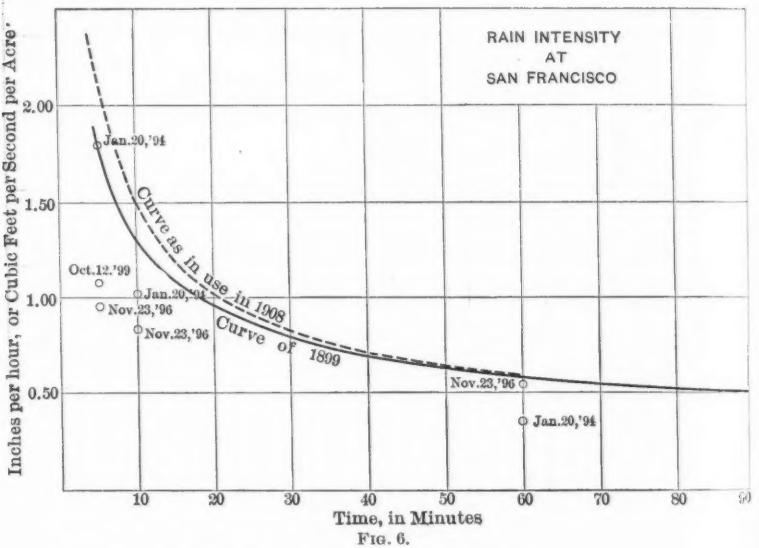

FIG. 6.

Table 2 is the rainfall rate table now in use.

TABLE 2.—RAINFALL INTENSITY AT SAN FRANCISCO, AS IN USE IN 1908.

Time, in minutes.	Intensity, in inches per hour.	Time, in minutes.	Intensity, in inches per hour.
5	2.16	20	1.02
6	1.97	30	0.83
7	1.81	40	0.72
8	1.69	50	0.65
9	1.59	60	0.60
10	1.50	70	0.56
15	1.20	..	....

In order that the great disparity between the maximum rainfall rates to be expected on the Pacific Coast and those of the Atlantic

Slope may become apparent, Fig. 4 shows several other limiting rain curves, in part taken from the report of the Board of Consulting Engineers for the City of Baltimore.* The limiting rain curve adopted by this board for Baltimore, and also a limiting curve determined by rainfall in Washington, D. C., in Philadelphia, Baltimore, and in the State of Maryland, are reproduced in the diagram, and also the limiting rain curves suggested by A. N. Talbot, M. Am. Soc. C. E., and by Emil Kuichling, M. Am. Soc. C. E.

The rainfall intensities indicated by the San Francisco curve of 1899 can be expressed by a formula of the type

$$I = \frac{b}{\frac{2\,t}{t+60} + t^{0.4}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (21)$$

Here I is the greatest possible mean rate of rainfall (the maximum intensity), expressed in inches per hour, when t represents the duration of the downpour, in minutes. The numerator, b, is a constant for any locality, but will vary within wide limits for different localities.

For San Francisco, $b = 3.68$.

For points on the Atlantic Slope, b will generally have a value lying between 8 and 12.

In Table 3 rain intensities are noted for several values of b, and, for comparison, the rain intensities as a deduced from the limiting rain curve of 1899 are also noted.

The Estimate of Storm-Water Flow.

Method of 1899.

Having determined the maximum rainfall rates, some method had to be adopted of estimating from these rates, the maximum rate of flow to be expected at any point of the storm-water conduits. Such method must of course be applicable to areas of all manner of shapes and sizes, of varied surface topography, and of varied character so far as determined by the improvements, present and prospective.

It is obvious that the maximum rate of flow at any point of a storm-water conduit will be less than the maximum rate of rainfall upon the tributary area. Some of the rain will be taken up by porous surfaces, some of it will evaporate, and at the time that the flow is a maximum much of the rain-water will still be scattered over the

* Report of Sewerage Commission, City of Baltimore, 1897.

surface or will be in the conduits the filling of which retards the passage of and elongates the flood wave.

TABLE 3.—MAXIMUM INTENSITY OF RAINFALL, AS DETERMINED IN 1899 FOR SAN FRANCISCO, AND BASED ON THE FORMULA:

$$I = \frac{b}{\frac{2\,t}{t+60} + t^{0.4}}$$

Duration of rain, in minutes	SAN FRANCISCO.			ATLANTIC SLOPE.	
	From curve of 1899. Inches per hour.	By formula:		By formula:	
		$b = 3.68$ Inches per hour.	$b = 4$ Inches per hour.	$b = 8$ Inches per hour.	$b = 12$ Inches per hour.
5	1 80	1.80	1.95	3.90	5.85
10	1.30	1.32	1.43	2.86	4.28
15	1.10	1.10	1.19	2.38	3.57
20	0.95	0.97	1.05	2.10	3.15
30	0.80	0.81	0.87	1.75	2.62
40	0.70	0.72	0.77	1.55	2.33
50	0.65	0.65	0.70	1.40	2.11
60	0.60	0.60	0.65	1.30	1.95
90	0.50	0.51	0.55	1.10	1.65
120	0.45	0.15	0.49	0.98	1.47
180	0.38	0.387	0.42	0.84	1 26
240	0.34	0.350	0.38	0.76	1.14
1 440	0.20	0.182	....	....	....

Experience in other cities indicates that the combined effect of these causes, which contribute to make the rate of flow in the conduit less than the rain rate, can with some degree of approximation be expressed as a percentage based upon the proportional part of the drainage area which is impervious. This fact has been pointed out by Mr. Emil Kuichling,* and also the further fact that density of population may be used as a guide in determining the correction factor to be used.

It was entirely out of the question to make experiments in San Francisco to be used in the approximation of reduction factors. The assumptions made, therefore, were based upon the experience in other cities. In San Francisco, as has been explained, the demarkation of the area tributary to any storm-water main is not possible without first modifying the sewer crossings. But even if it had been possible to prepare one or more areas for observation, it would have been folly to do

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this in the hope that some storm might occur within the short time available for preparing the report, particularly as results were wanted before the next wet season. It is to be remembered, too, that such results would have been of slight value, because they would have represented run-off rates for conditions as they prevailed in 1899 and not for those that will prevail in the future.

The study of population distribution was based largely upon the records in the Registrar's office. From the size of voting districts and the number of votes in each, with due allowance for the class of people inhabiting each, an estimate of population density was made. Based on this study, it seemed proper to regard all parts of the city having a population of 100 or more per acre as impervious. A population density was then assumed for each sewer district such that the aggregate in all districts would be 1 000 000.

Table 4 shows the reduction factors finally used in estimating the required storm-water capacity from the rate of rainfall.

TABLE 4.

Population per acre.	Reduction factor.	Population per acre.	Reduction factor.
20	0.30	70	0.65
30	0.40	80	0.70
40	0.50	90	0.73
50	0.55	100	0.75
60	0.60	...	....

The procedure in determining the required storm-water capacity for any point of a conduit was substantially as follows: The time was approximated which would elapse before the water falling as rain in the most remote part of the district would reach the point at which the capacity was to be determined. Distance and fall between governing points along drainage lines enabled a quick and sufficiently close approximation of this time to be made. From 3 to 5 min. were then added as the time allowance for the rain-water to collect in gutters and flow to the storm-water inlets. Although within this elapsed time, there may have been shorter periods with higher rates of rainfall, it is to be assumed that these were practically coincident throughout the district, and that much of the water falling at the maximum rate upon nearby portions of the area had already passed before the water from more remote sections arrived. Consequently, the average rate of

rainfall during the entire critical period was alone taken into account. This was the maximum rate of rain for the estimated time, as shown by the curve in Fig. 6.

The reduction factor, determined by population density, or otherwise, was then applied. In this way the maximum run-off rate per acre was estimated from point to point for each main sewer. Where two main lines of sewers came together, independent estimates were made for each, and the results were combined, if the two districts were not too dissimilar in shape and extent. Otherwise, the time determination, as made for the larger district alone, was used in calculating the rate of run-off. This was done because the maximum flow in the drainway of the smaller district would occur earlier than in the larger district and the allowance for required capacity might become unreasonably large.

It is to be understood, that this method of calculating storm-water flow was applied with care. Cases are readily conceivable where the shape of a contributing area is such that it would be improper to apply to the entire area a rain rate determined from the time required by water to flow from the most remote point thereof. The area under consideration may have one or more finger-like projections, long and narrow, which, if taken into account, would make the time long and the rain rate small. In such cases it becomes necessary to examine the lower compact portion of the area by itself. The rain rate is determined in the usual way, and if its application to the partial area, particularly if, owing to surface character, a larger proportion of run-off is to be assumed, gives larger results than the treatment of the district as a whole, then this value determines the required conduit capacity for storm-waters. Sometimes a succession of trials is requisite to establish the maximum possible value.

This method, in its entirety, however, is unsatisfactory, mainly for the reason that it leaves too much latitude to the judgment of the engineer. It has not found general acceptance.

Similar and yet more serious objection may be urged against the use of the various formulas for storm-water flow that have from time to time been suggested by hydraulicians for application to urban areas. None of these, moreover, could have been applied to San Francisco conditions without special study relating to the determination of constants to fit them to local conditions. Even then there would have been grave doubt as to whether formulas predicated on rainfalls exceeding

those at San Francisco from two to six-fold for time periods of one hour and less should be considered applicable. It is only necessary to glance at the rain curves in Fig. 4 to appreciate the force of this statement.

In all the formulas that have come to notice the rate of rain with some suitable exponent, generally unity, appears as a factor. The rain rate to be used in the formulas must be determined by the engineer, whose judgment is guided by the available rainfall data and his past experience. Thereupon a correction factor, based upon the character of the surface of the district, must be adopted. Even when rain data are ample and the constants have been determined with care the result may be rendered more or less doubtful if the district under consideration has an unusual shape, or has exceptional hypsographic features.

These considerations led to the conclusion, in 1899, that it was not practicable to adapt any of such well-known formulas as those of Hawksley, Adams, McMath or Bürkli-Ziegler, to the conditions prevailing in San Francisco.

The method already described, appearing in 1899 to be the most rational then known, was therefore adopted.

But the need was felt, and has not heretofore been met, of some more satisfactory method of translating rain rates into storm-water flow. The result of a further study of this subject can now be presented with the hope that it may prove of some value to the Engineering Profession.

New Method of Estimating Required Capacity of Storm-Water Conduits.

Surface Run-off.—The rain which falls upon any area during the time in which the run-off rate is increasing is in part accounted for by infiltration into porous soils and by evaporation; in part it flows past the outfall point of this area, and in part it has swelled the water contents of the storm-water conduits within the area, or is still in transit across the surface to the storm-water inlets.

It has been stated that the method of estimating sewer capacities, as used in the San Francisco problems, takes into account, after a fashion, the retarding influence of temporary water storage on the ground and in the conduits. The new method makes this storage,

which is capable of approximation, an essential element in the calculation of storm-water flow.

This method is applicable wherever it is possible to determine the volumetric increase of the water actually in the conduits during the progress of a rain of the extreme type. The first step, again, is the determination of the maximum rain rates for increasing time intervals. Thereupon, with these rain rates as a guide, the rates can be determined at which the water will run from the surface of the ground. The final step is the modification of these surface run-off rates by the effect of water accumulation or storage on the surface and within all conduits which are located above the point at which storm-water capacity is to be determined.

It is to be remembered that the percentage correction, heretofore noted as applicable to rain rates when storm-water flow is to be estimated by the method used in 1899, covers, not alone the reduction of flow due to water absorption and evaporation, but also the retarding effect due to temporary water storage. These correction factors, therefore, cannot be used when it is desired to estimate for a relatively short time during the progress of a heavy rainfall how much of the water reaches and enters the storm-water inlets.

The greatest intensity of rain is rarely at the beginning of a storm. For the purpose of estimating conduit capacities it must be assumed to occur at a time when the ground is already wet and no longer capable of absorbing water as rapidly as at the beginning of the rain. The rate of water absorption by porous ground plus evaporation though possibly several times greater at the commencement of a downpour than after the same has continued for some time, by no means keeps pace with the rain rate. It follows that it would not be correct to apply a reduction factor, ascertained from aggregate quantities of rain and of run-off produced by the storm, to the maximum rain rate in order to estimate therefrom the maximum surface run-off rate.

The amount of water which evaporates during the short time covered by the critical periods involved in urban run-off problems, will be relatively small and, for all practical purposes, may be disregarded.

The run-off, consequently, from an impervious area, the highest supposable type of urban area, during any critical period, measured on the ground where the rain falls, may be regarded as equal to the

maximum rain intensity for that period; and the rate of inflow into storm-water inlets will be this run-off rate, modified only by the effect of changes in the volume of water temporarily stored on the surface of the area.

It will be shown, hereinafter, that the amount of water actually lying upon or flowing over the exposed surfaces of the area under consideration cannot be materially different, at the end of a critical time period, from the amount thus covering the area at the beginning of the period, except when the critical time period is of short duration. It may be neglected whenever the critical time period exceeds 15 min.

The amount of water in temporary storage on the ground's surface, that is, the water momentarily in transit, on roofs, on lawns, in gutters and otherwise, to the storm-water inlets, can be estimated if the time is known that it will take water during heavy rains to flow to the inlets from the outer portions of the small areas served by the individual storm-water inlets. This time is usually from 5 to 10 min. Make the unfavorable assumption that it is 5 min.

The area tributary to each inlet may now be regarded as subdivided into 5 concentric zones, from the innermost of which water will reach the inlet in the average time of ½ min., from the next in 1½ min., and so on to the last one in 4½ min.

At the end of any 5 min., supposing the rain to fall at a uniform rate during this time, there will still be on the ground, flowing toward the inlet $\frac{4.5}{5}$ in. of the rain which fell on the outer zone, $\frac{3.5}{5}$ in. of the rain which fell on the next smaller zone, and so on down to the innermost zone, on which there will still be one-half of the rain which fell during the last minute. (No correction has been made for the fact that the outer half, by time, of each zone is somewhat larger than the inner half.)

If, now, the area which is served by an inlet be considered as circular, with the inlet in the center of the area, then the areas of the successive zones and the entire area will be to each other as 4, 12, 20, 28, 36, and 100.

Calling the total amount of rain which fell upon the area in 5 min. $100x$, then the amount of water remaining on the ground at the end of this period will be

$$\left(\frac{4}{10}+\frac{36}{10}+\frac{100}{10}+\frac{196}{10}+\frac{324}{10}\right)x = 66x$$

The water on the ground at the end of the 5-min. period, therefore, is $\frac{66\,x}{100\,x}$, or two-thirds of the total amount of rain which fell in that period.

Generalizing, it may be said that what is true of an area with circular outline is true of other compact shapes, and that, therefore, two-thirds of the water which falls during any time period equal to the time required by water to reach the inlets from the outer portions of areas drained by each, will, at the end of that period, be still on the surface in transit to the inlet.

For any limiting rain curve, therefore, the amount of water on the ground in a district with impervious surface can be estimated for any instant of time, if the time is known that will be consumed by the water in flowing from the limits of an inlet area to the inlet.

For the inlet or entrance time equal to 5 min., and the rain rates indicated by the limiting rain curve of 1899 for San Francisco, the amount of water on the ground at the end of successive intervals of 5 min. is shown in Column 5 of Table 5.

TABLE 5.—WATER ON THE GROUND DURING RAINFALLS OF EXTREME INTENSITY AT SAN FRANCISCO.

Based on Rain Intensities as determined in 1899, using the formula:

$$I = \frac{3.68}{\frac{2\,t}{t+60} + t^{0.4}}$$

Successive periods: Duration, in minutes.	Greatest possible total rain to the end of the period, in inches.	Rain during each successive period, in inches.	Intensity of rainfall, in inches per hour; or, approximately, second-feet per acre.	Water on the ground at the end of each period, in cubic feet per acre.	Greatest possible total rain to the end of each period, in cubic feet per acre.	$G_T - G_{T+5}$ cubic feet per acre.
(1)	(2)	(3)	(4)	(5)	(6)	(7)
0 to 5	0.150	0.150	1.795	360	540	....
5 to 10	0.219	0.069	0.828	166	790	195
10 to 15	0.275	0.056	0.672	134	1 000	32
15 to 20	0.321	0.046	0.552	112	1 170	22
20 to 25	0.364	0.043	0.516	103	1 320	9
25 to 30	0.404	0.040	0.480	95	1 470	8
30 to 35	0.441	0.037	0.444	89	1 610	6
35 to 40	0.476	0.035	0.420	84	1 730	5
40 to 45	0.509	0.033	0.396	79	1 830	5
45 to 50	0.541	0.032	0.374	75	1 970	4
50 to 55	0.571	0.030	0.360	72	2 080	3
55 to 60	0.600	0.029	0.348	70	2 180	2
60 to 120	0.906	0.306	0.306	61	3 290	....
120 to 180	1.158	0.252	0.252	50	4 200	....
80 to 240	1.396	0.238	0.238	48	5 090	....

TABLE 6.—REDUCTION FACTORS WITH WHICH TO MULTIPLY MAXIMUM RAIN RATES TO FIND MAXIMUM SURFACE RUN-OFF RATES.

The value of the constant, a, in $r = a\,I$

$$\text{When } I = \frac{b}{\frac{2\,t}{t+60} + t^{0.4}}$$

Duration of the critical period, in minutes.	San Francisco. $b = 3.68$ $R = 0.60$ $e = 0.0775$					$b = 4$ $R = 0.65$ $e = 0.084$					$b = 8$ $R = 1.30$ $e = 0.169$					$b = 12$ $R = 1.95$ $e = 0.253$				
	All impervious.	75% impervious.	50% impervious.	25% impervious.	None impervious.	All impervious.	75% impervious.	50% impervious.	25% impervious.	None impervious.	All impervious.	75% impervious.	50% impervious.	25% impervious.	None impervious.	All impervious.	75% impervious.	50% impervious.	25% impervious.	None impervious.
5	1.00	0.91	0.81	0.71	0.61	1.00	0.91	0.83	0.73	0.64	1.00	0.96	0.91	0.87	0.82	1.00	0.97	0.94	0.91	0.88
10	1.00	0.88	0.76	0.64	0.51	1.00	0.89	0.77	0.66	0.54	1.00	0.94	0.88	0.83	0.77	1.00	0.97	0.93	0.89	0.85
15	1.00	0.86	0.71	0.57	0.42	1.00	0.87	0.74	0.60	0.47	1.00	0.93	0.87	0.81	0.74	1.00	0.96	0.92	0.87	0.83
20	1.00	0.84	0.68	0.53	0.37	1.00	0.86	0.72	0.56	0.43	1.00	0.93	0.86	0.79	0.72	1.00	0.96	0.91	0.86	0.81
30	1.00	0.83	0.66	0.49	0.31	1.00	0.85	0.69	0.53	0.37	1.00	0.92	0.84	0.76	0.69	1.00	0.95	0.90	0.85	0.79
40	1.00	0.82	0.64	0.46	0.27	1.00	0.84	0.67	0.51	0.34	1.00	0.92	0.83	0.75	0.67	1.00	0.95	0.89	0.84	0.78
50	1.00	0.81	0.63	0.44	0.25	1.00	0.83	0.66	0.49	0.31	1.00	0.91	0.83	0.74	0.66	1.00	0.94	0.88	0.83	0.77
60	1.00	0.81	0.62	0.43	0.23	1.00	0.82	0.65	0.47	0.29	1.00	0.91	0.82	0.73	0.65	1.00	0.94	0.88	0.82	0.76
90	1.00	0.80	0.59	0.38	0.18	1.00	0.82	0.63	0.44	0.25	1.00	0.91	0.82	0.72	0.63	1.00	0.94	0.88	0.81	0.75
120	1.00	0.79	0.58	0.37	0.16	1.00	0.81	0.62	0.42	0.23	1.00	0.91	0.81	0.71	0.62	1.00	0.94	0.87	0.80	0.74

The absorption of water by the soil will vary directly with the proportional part of the surface that is pervious, and will also vary with the character of the soil and subsoil. It will be largest in districts having deep soils of the sand or gravel type that take an indefinite quantity of water freely, and will be least in districts with heavy clay soil or with a thin soil cover on impervious rock formations.

The rate at which the water thus deflected from a course across the surface of the ground will sink into the soil should be assumed to be a gradually decreasing rate. Soil will take water somewhat more readily at the beginning of a downpour than after a heavy rain has continued for some time.

This rate is determinable, and upon it will depend the relative amount of the rain which will from time to time, during the progress of a heavy rain, reach the storm-water inlets.

Data have not been accessible for an entirely satisfactory determination of reduction factors to be applied to rain rates to reduce the rain rates to surface run-off rates. The factors noted in Table 6, therefore, are to be considered as tentatively suggested. They are to be used until checked and corrected by further observations. They apply to soil conditions such that the rate of water absorption by the soil at the beginning of a critical period will be about $\frac{2}{3}$ in. per hour, decreasing gradually to about $\frac{1}{3}$ in. per hour at the end of 1 hour.

Notation.—The following notation is used in this paper:

A = Area, in acres, of the district above the point at which storm-water discharge is to be estimated;

T = End of a period of heavy rainfall;

T_m = Time at which the storm-water discharge is a maximum;

B = Beginning of a period of heavy rainfall;

B' = Time, t minutes preceding B;

t = Elapsed time, in minutes, from B to T;

t_m = Elapsed time, in minutes, from B to T_m;

R = Maximum rainfall in one hour; expressed in inches;

I = Maximum rain rate, in inches per hour (or, which is the same, second-feet per acre), for a period of time, t minutes in duration;

I_B = Maximum rain rate, in inches per hour, during t minutes preceding the time, B, during time, B' to B;

u_T = The rate of inflow into storm-water inlets during the time, B to T, expressed in cubic feet per second per acre;

u_B = The rate of inflow into storm-water inlets during the time, B' to B;

C = Volumetric capacity for storm-water (storage capacity) of all conduits, above the point at which discharge is to be estimated, expressed in cubic feet per acre;

C_B = Storm-water contents of the conduits at the time, B, expressed in cubic feet per acre;

C_T = Storm-water contents of the conduits at the time, T, expressed in cubic feet per acre;

$C_T - C_B$ = Therefore the augmentation of the water temporarily stored in the conduits during the time, B to T;

G_B = The water on the ground, on roofs, etc., and in the gutters, flowing toward the storm-water inlets, at the time, B, expressed in cubic feet per acre;

G_T = The water on the ground, etc., flowing toward the storm-water inlets at the time, T;

G_{T+i} = The water on the ground, etc., flowing toward the storm-water inlets, at the end of i minutes, during which the rain intensity is the greatest possible, immediately following a critical period the duration of which is t minutes;

r = The surface run-off rate during the time, B to T, expressed in depth, in inches per hour, or in cubic feet per second per acre;

r' = The surface run-off rate during the time, B' to B, expressed in depth, in inches per hour, or in cubic feet per second per acre;

a = The constant in the equation $r = a\,I$ (and approximately $r' = a\,I_B$). This constant will have different values for different areas. For areas with impervious surface throughout, a is unity;

b = A constant appearing in the formula for rain intensity;

d = The mean discharge of storm-water, in cubic feet per second per acre, at the point for which storm-water discharge is to be estimated, during the time, B to T;

d_T = The storm-water discharge at the time, T, expressed in cubic feet per second per acre;

d_m = The maximum storm-water discharge at the time, T_m, expressed in cubic feet per second per acre;

d_B = The storm-water flow at the time, B, expressed in cubic feet per second per acre;

t_D = The time, in minutes, required for water to flow from the most remote portions of a district to the point at which discharge is to be estimated (including a time allowance, i, for flow to inlets);

i = The time, in minutes, required by water to flow from the outermost portions of the area served by a storm-water inlet to the inlet;

S = The increase of the water quantity in temporary storage on the ground and in the conduits during the time, B to T;

S_m = The increase of the water quantity in temporary storage on the ground and in the conduits during the time, B to T_m;

q = The total depth of rain, in inches, during the time, B to T;

q' = The surface run-off, in inches, during the time, B to T;

e = A constant, appearing in the special case when the limiting rain curve is a parabola.

The Storm-Water Flow.—The maximum quantity of rain, expressed in cubic feet, that may fall in the time, B to T, on one acre, will be $60\ I\ t$. The portion of this water which will reach the storm-water inlets is $60\ r\ t$. This is the surface run-off per unit area, and must be equal to the total flow during the time, B to T, at the point for which discharge is to be estimated, plus the augmentation of the water storage on the ground and in the conduits during the same time.

$$60\ r\ t = 60\ a\ I\ t \quad \ldots\ldots \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

$$60\ r\ t = 60\ d\ t + (C_T - C_B) + (G_T - G_B) \quad \ldots\ldots\ldots (2)$$

$$d = r - \frac{(C_T - C_B) + (G_T - G_B)}{60\ t} \quad \ldots\ldots\ldots\ldots (3)$$

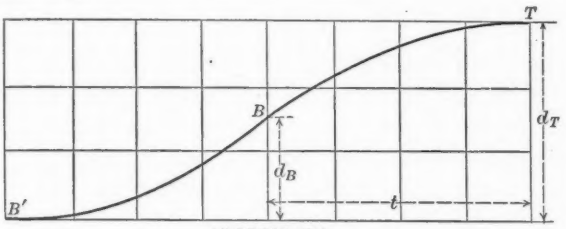
$$\text{or} \quad d = a\,I - \frac{(C_T - C_B) + (G_T - G_B)}{60\ t} \quad \ldots\ldots\ldots\ldots (4)$$

But if d is known, and if d_B can be determined, it will be possible to find the value of d_T on the assumption that

$$d = \frac{2}{3} d_T + \frac{1}{3} d_B \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (5)$$

$$\text{From which} \quad d_T = \frac{1}{2} (3\,d - d_B) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (6)$$

The value assumed for d in Equation 5 results from the hypothesis that the hydrograph for the time, B to T, approximates in shape a parabola, tangent to a horizontal line at T and passing through the point B, as shown in Fig. 7.



HYDROGRAPH

FIG. 7.

But the value of d_B can be approximated on the further assumption that only the rainfall in the t_m minutes preceding the time, B, need be taken into account. For the greatest possible rain intensity preceding B, the value of d_B, without too great error, may be called equal to the mean run-off rate during the t_m minutes preceding B. That is to say:

Because, approximately, $r' = a\,I_B$ ..... (7)

Therefore, near enough, $d_B = r' = a\,I_B$ ..... (8)

And from Equation 6, $d_T = \frac{1}{2}(3\,d - a\,I_B)$ ..... (9)

It appears from the above that the value of d increases with increasing values of C_B. It increases in value, also, for decreasing values of C_T. (See Equation 4.) Therefore the rain which will give d maximum value must be so distributed to the time preceding and immediately following the time, B, that the conduits will be as full of water as possible at the beginning, and at as low a stage as possible at the end, of the critical period. With this fact the type of rain storm becomes known which will make the mean discharge throughout a critical period a maximum.

The rain must evidently fall preceding the time, B, with constantly increasing intensity, and during the time, B to T, with constantly decreasing intensity.

It follows, that the greatest possible rain intensity, at a time instant, immediately preceding B, will be the same as the greatest possible rain intensity at the close of a critical period. Consequently, for the most unfavorable rain conditions, the value of G_B will be equal to G_{T+i} (the inflow time being i minutes).

Equation 3 may now be written

$$d = r - \frac{(C_T - C_B) + (G_T - G_{T+i})}{60t} \quad \ldots\ldots\ldots (10)$$

but $$S = C_T - C_B + G_T - G_{T+i} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (11)$$

therefore $$d = r - \frac{S}{60\,t} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (12)$$

The values of G_T and G_{T+i} can be calculated for any rain curve, as has already been shown. The value of d in Equations 3 and 10 and of d_T in Equation 6 are dependent, not alone upon the intensity of the rain, but also upon the duration of the extreme rain conditions, and upon the volumetric increase of the water in the conduits and on the ground during the critical period.

It may be stated in passing that, for the inlet ends of the conduits, $C_T - C_B = 0$. Therefore, from Equation 10, for the special case in which $d = u$ and $d_T = u_T$

$$u = r - \frac{G_T - G_{T+i}}{60\,t} \quad \ldots\ldots\ldots\ldots\ldots\ldots (13)$$

calling $i = 5$

$$u = r - \frac{G_T - G_{T+5}}{60\,t} \quad \ldots\ldots\ldots\ldots\ldots\ldots (14)$$

and from Equation 6

$$u_T = \frac{1}{2}\,(3\,u - r') \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (15)$$

It may be inferred from Equation 9,

$$d_T = \frac{1}{2}\,(3\,d - a\,I_B),$$

that the value of d_T will follow closely the value of d; that, therefore, the same type of rain storm which will give the mean discharge during a critical period maximum value will also make the discharge at the time, T, maximum. It is reasonably certain that this is the case, as will be shown later in the presentation of a graphical solution of the problem. This fact is not demonstrated by Equation 9, because C_B is at maximum value when I_B is at maximum value, while d_T increases with the increasing value of C_B and with the decreasing value of I_B. The graphical illustration will show that in all ordinary cases the effect of changing values of C_B upon the value of d_T outweighs the effect of the corresponding changes in the rain rate, I_B. But the introduction of the variable, I_B, into the calculation of d_T from d makes it apparent that the time at which d is maximum is not

necessarily the time at which d_T is maximum. These times, in fact, are not coincident.

The least satisfactory part of the foregoing demonstration lies in the approximation of a value for d_B and in the assumption that the maximum discharge can be computed from the mean discharge during the critical period and the discharge at the beginning of the period.

Nor is it, as a matter of course, strictly correct, except in the case of impervious areas, to assume that the relation of r' to I_B is the same as the relation of r to I. This assumption is probably near enough to the truth for all practical purposes.

The true value of d_T, therefore, may depart to some extent from the value determined, as explained, from d and d_B. The room for such departure will decrease as the values of d_B and d approach each other; it will be greatest when $d_B = 0$; that is, the opportunity for a departure from the correct value is greatest in the case of no rain preceding the critical period. It is quite possible that for this special case some other expression than Equation 9 can be found that will better express the relation between d, d_B, and d_T; but the further investigation of this special case would only be of interest if it were desired to deduce formulas for application to types of rain storms other than those which produce the maximum discharge.

To estimate the values of C_T and C_B, let it be assumed that a first approximation of conduit sizes by any method has established a first value of C. The value of C when the conduits are to carry water other than that due to rain will not, as a matter of course, be the entire volumetric contents of the conduits, but that portion thereof which is available for the temporary storage of rain water. Call t_m equal to t_D. This fixes tentatively the position of T in the time scale. The limiting rain curve furnishes the value of I and of I_B, and, by application of the proper reduction factor, the values of r and of r' are found from $r = a\,I$ and $r' = a\,I_B$.

It is now possible to estimate the maximum value of C_B. This will result, as already explained, if the rain storm is of such a character that, preceding the time, B, there is a rainfall of constantly increasing intensity. See Fig. 7. The flow at the time, B, can now be calculated; but the ratio of this flow to the flow at the time, T, may be, preliminarily, considered the same as the ratio of I_B to I. With the aid of this ratio, on the assumption that the space occu-

pied by water in the conduits is proportional to the flow (the velocities being practically the same for half full and for full or nearly full stages) a first approximation of C_B is possible. The first approximate value of C_T is C. That is, all conduits are assumed to be full at the time, T.

The value of d resulting from this assumption will be too small, because, even under the most unfavorable conditions, C_T will always be somewhat less than C.

The value of d_T follows by the use of Equation 9.

The conduit sizes originally entering into the calculation can now be revised, and the values of C_B and C_T are again determined. It is now desirable to make the estimate of C_T with greater precision than for first approximation purposes. This can readily be done, because, at the time, T, the conduits are full at the point for which storm-water flow is to be estimated and the rate of flow at all upper ends of conduits can be estimated from the rain rate which prevailed in the last i minutes preceding the time, T.

It will ordinarily be found convenient to call

$$C_T = KC \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (16)$$

where K is a coefficient to be ascertained for each area under consideration. The value of K will approach unity for small districts.

From Equations 9 and 10,

$$d = r - \frac{K\,C - C_B}{60\,t} - \frac{G_T - G_{T+5}}{60\,t}$$

and $$d_T = \frac{1}{2}\left[3\left(r - \frac{K\,C - C_B}{60\,t} - \frac{G_T - G_{T+5}}{60\,t}\right) - r'\right]$$

For all cases in which t exceeds 15 min., $G_T - G_{T+5}$ will have so small a value that the term containing this expression may be neglected, therefore $t > 15$

$$(d_T)_{t>15} = \frac{1}{2}\left[3\left(r - \frac{K\,C - C_B}{60\,t}\right) - r'\right] \quad \ldots\ldots\ldots\ldots (19)$$

or $$(d_T)_{t>15} = \frac{1}{2}\left[3\left(a\,I - \frac{K\,C - C_B}{60\,t}\right) - a\,I_B\right] \quad \ldots\ldots\ldots (20)$$

Numerical Illustrations.—Let it be required to determine the capacity of the outfall sewer for a business district, with impervious surface throughout, in San Francisco, the area of the district being 1 000 acres. Suppose that the district has such shape and such surface slope that it will take water 50 min., including a 5-min. entrance allowance, to flow from the most remote portion thereof to the point

at which the conduit capacity is to be determined. Temporary storage of water on the ground's surface, because $t_D > 15$, may be neglected ($G_T - G_{T+5} = 0$).

The maximum rain rate in San Francisco for 50 min., when $I = \frac{3.68}{\frac{2\,t}{t+60} + A^{0.4}}$ (see Table 2), is 0.65 in. per hour, or 0.65 sec-ft. per acre.

For an impervious surface, $a = 1.00$; therefore $r = I$, and $r' = I_B$.

A first approximation of the storm-water discharge can be made by the method heretofore described as in use in San Francisco.

By this method,

$$d_m = 0.75 \times 0.65 = 0.488 \text{ sec-ft. per acre.}$$

Let it now be assumed that calculations for a number of points in the district have established conduit sizes having a volumetric capacity for storm-water of 1 500 cu. ft. per acre.

Let it further be supposed that calculations already made have established $K = 0.8$.

An examination of the limiting rain curve of the type recommended for use in San Francisco will show that during a period of time, t minutes in duration, immediately preceding or following a critical period of the same duration, the greatest possible rainfall will be, in round numbers, one-half of the rain during the critical period. For approximation purposes, therefore, the quantity of water in the conduits at the beginning of a critical period may be taken at one-half of the amount in the conduits at the close of the period.

Approximately $\quad C_B = \frac{C_T}{2},$

but $\quad C_T = K\,C = 0.8\,C,$

therefore, $\quad C_T - C_B = 0.4\,C = 600.$

The limiting rain curve for San Francisco, based on $I = \frac{3.68}{\frac{2\,t}{t+60} + t^{0.4}}$, Fig. 6, indicates that in the 50 min. preceding the time, T, there may be a rainfall aggregating 0.531 in.; that is, $I = 0.649$ in. per hour. The rainfall in 50 min. preceding the critical period cannot, as shown by the rain curve and Table 2, exceed 0.261 in., because the total rain in 100 min. cannot exceed 0.792 in.

The rainfall, 0.261 in. in 50 min., represents a rain intensity of 0.313 sec-ft. per acre.

Consequently, $r = a\,I = \quad 0.649$ (a being unity)

$$r' = a\,I_B = 0.313$$

From Equation 19, $$d_T = \frac{1}{2}\left[3\left(0.649 - \frac{600}{3\,000}\right) - 0.313\right]$$

$$= 0.517 \text{ sec-ft. per acre.}$$

$$A\,d_T = 517 \text{ sec-ft.}$$

As this value is somewhat larger than the first approximation, it may now be necessary to correct the value of C. For the purpose of this illustration, it will be supposed that no correction is necessary.

Other values of t are now to be tried, in order to find that value of t which will give d_T maximum value.

For $t = 40$ min.

$$r = 0.714$$

$$r' = 0.342$$

$$d_T = \frac{1}{2}\left[3\left(0.714 - \frac{600}{2\,400}\right) - 0.342\right]$$

$$= 0.525$$

$$A\,d_T = 525 \text{ sec-ft.}$$

For $t = 30$ min.

$$r = 0.808$$

$$r' = 0.390$$

$$d_T = \frac{1}{2}\left[3\left(0.808 - \frac{600}{1\,800}\right) - 0.390\right]$$

$$= 0.468$$

$$A\,d_T = 468 \text{ sec-ft.}$$

The critical period is evidently about 40 min., and the maximum storm-water discharge, or required conduit capacity, will be about 525 sec-ft.

As a second illustration, let it be required to ascertain the storm-water discharge from a district, 1 000 acres in area, in which only 250 acres of the surface are impervious. Here, again, the time, t_D, is to be approximated from probable conduit gradients. Suppose $t_D = 50$ min.

The value of a, to be used in the equations $r = a\,I$ and $r' = a\,I_B$, will be found in Table 6, that is $a = 0.44$.

Suppose that, by preliminary estimate, $C = 800$, and that $K = 0.8$.

Again accepting the approximate value, $C_B = \frac{C_T}{2}$, makes $C_B = 320$.

For $t = t_D = 50$ min.

$I = 0.649$

and $I_B = 0.313$

$r = 0.44\, I = 0.285$

$r' = 0.44\, I_B = 0.138$

$$d_T = \frac{1}{2}\left[3\left(0.285 - \frac{320}{3\,000}\right) - 0.138\right]$$

$$= 0.198$$

$A\, d_T = 198$ sec-ft.

For $t = 40$ min.

$I = 0.714$

$I_B = 0.342$

$r = 0.44\, I = 0.315$

$r' = 0.44\, I_B = 0.151$

$$d_T = \frac{1}{2}\left[3\left(0.315 - \frac{320}{2\,400}\right) - 0.151\right]$$

$$= 0.197$$

$A\, d_T = 197$ sec-ft.

For $t = 60$ min.

$I = 0.600$

$I_B = 0.291$

$r = 0.44\, I = 0.264$

$r' = 0.44\, I_B = 0.128$

$$d_T = \frac{1}{2}\left[3\left(264 - \frac{320}{3\,600}\right) - 0.128\right]$$

$$= 0.184$$

$A\, d_T = 184$ sec-ft.

The critical time is evidently about 50 min., and the required capacity about 198 sec-ft.

It must be remembered that the values of the reduction factor a noted in Table 6 are based on a specific rate of water absorption by the soil. They are to be suitably modified whenever, for any district, other rates of water absorption by the soil shall have been determined.

The increase of water contents of the conduits, when conduits are already constructed, should be calculated from the estimated water stages whenever this is practicable, instead of using the above indicated approximation method.

How the value of t_m may be found, when the storage increase during a critical period is known, will be explained later.

Graphical Solution.—The method of estimating the required capacity of storm-water conduits just illustrated becomes of importance, first, because it is based on sound principles and can be made applicable to districts of any shape, character, and size; and second, because it enables the problems to be solved by simple graphical methods, as will now be explained.

With the mass curve of rainfall for extreme conditions of precipitation as a guide, there is to be constructed for any area under consideration a second integral or mass curve representing the maximum amount of surface run-off during the progress of a downpour of the extreme type. The new curve will be the rain curve modified by the appropriate reduction factor, a, to be found in Table 6. The new curve, for convenience, may be called the surface run-off curve.

The surface run-off curve for areas with impervious surface throughout will coincide with the limiting rain curve, because, for the impervious surface, the factor, a, is unity and $r = I$. The San Francisco surface run-off curve for impervious surface, based on rain intensities expressed by the formula,

$$I = \frac{3.68}{\frac{2\,t}{t+60} + t^{0.4}}$$

(which will closely approximate the rain curve of 1899), is shown in Fig. 9.

The time scale in this diagram indicates minutes subsequent to the time, B. The ordinates represent, in cubic feet per acre, the water which will ultimately reach the storm-water inlets.

The total run-off during any time period is the difference between the ordinates at the beginning and at the end of the period. Some of this water, however, is still flowing across the surface toward the inlets at the end of the period.

The surface run-off curve for an area having a surface which is not impervious throughout can be obtained from the curve for impervious areas by multiplying the ordinates by the proper reduction factors.

Suppose now that the limiting surface run-off curve be platted in a system of co-ordinates with the point, B, at the beginning of the co-ordinates (Fig. 8).

Let T be located on this run-off mass curve so that B to T on the

time scale is the actual critical period for the point of the conduit under consideration.

Make $T D' = S$

the value of S being $S = C_T - C_B + G_T - G_{T+t}$ ..... (11)

then $d = \dfrac{D' F}{60\, t}$

The storm-water discharge at the time, B, as already explained, which will make d a maximum, will be the discharge due to a maximum possible fall of rain in the time, t minutes, preceding B. The rainfall in this time must be one of increasing intensity. The run-off curve, therefore, will take the position as shown, from B'' to B.

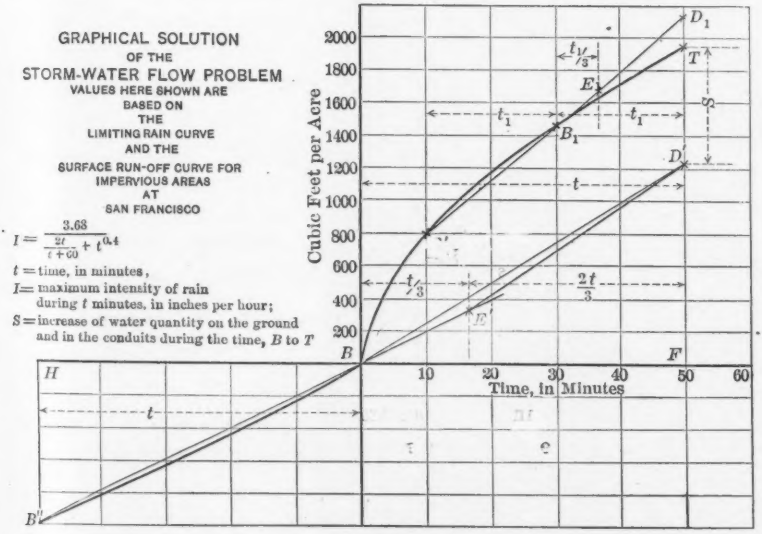


FIG. 8.

The discharge of storm-water at the time, B, may be taken, without material error, as equal to the mean surface run-off rate during the time, B'' to B, provided, of course, that t be not at material variation from t_D. Therefore,

$$d_B = \frac{B'' H}{60\, t}.$$

The mass curve of storm-water discharge after the time, B, will therefore be represented by a curve to which the line, $B''\, B$, is tangent at B, and which will gradually rise to the point, D'. The inclination

of the tangent to this curve at the point, D', will represent the maximum flow at the time, T.

The shape of this curve will probably be such that its change of direction is somewhat more rapid at the time, B, at which the downpour is most intense, than at the time, T, at which the flat top of the flood wave in the conduit has reached the point for which discharge is being estimated. It will be reasonable to assume, if B'' B be

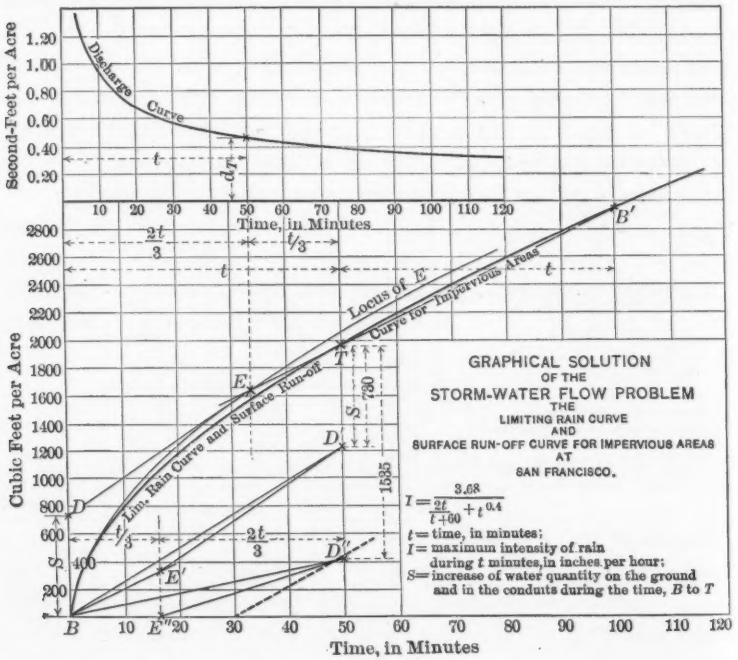


FIG. 9.

prolonged to the point, E', on the time ordinate, one-third of the way from B to T, that the line, E' D', will be a close approximation to the inclination of the curve at the point, D'. This is in conformity with the assumption heretofore made that

$$d = \frac{2\,d_T}{3} + \frac{d_B}{3} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (5)$$

By assuming a new position for the point, T, with corresponding modification of the curve, B'' to B, a new position of the point, E', will be found. It thus becomes possible, by determining successive locations of the points, E' and D', to find the particular value of t

which will give the line, E' D', its greatest inclination; which will, in other words, for the known value of S, give d_T maximum value.

The same result may be obtained in a much simpler way. Suppose the value of S to be scaled off upward from B in the diagram, Fig. 9, establishing the point, D. Let B' be the point on the run-off mass curve for a time period equal to 2 t. Then the line, B' T, of this diagram will be parallel to the line, B'' B, of the diagram, Fig. 7. If this line, B' T, prolonged cuts the time ordinate lying two-thirds of the way from B to T at E, then D E will be parallel to the line, E' D', of the diagram, Fig. 8, and the inclination of the line, D E, will give the desired value of d_T.

For every possible location of T a new position of E will be found. The successive positions of E will lie upon a curve which, as will readily be seen, is always the same no matter what the value of S may be. Therefore, to find the maximum value of d_T, it will only be necessary to draw a tangent from the point, D, Fig. 9, to the E-curve.

The point of tangency thus found will be distant from B on the time scale two-thirds of the time, t_m; and the inclination of the tangent will be the greatest possible value of d_T.

For the limiting rain curve of San Francisco, based upon

$$I = \frac{3.68}{\frac{2\,t}{t+60} + t^{0.4}}$$

and impervious surfaces ($a = 1$), the resulting locus of the point, E, or the E-curve, is shown in Fig. 9.

Suppose, now, that a value of $C_T - C_B$ has been approximated. Add to this $G_T - G_{T+5}$, obtained from Table 5, on the supposition that $i = 5$ and that t_m will be about equal to t_D

Scale upward from B the value of S thus obtained, and draw D E tangent to the E-curve at E.

Then d_T = inclination of line, D E,

and t_m = one and one-half times the scaled time from B to E.

It will be convenient to complete the diagram by platting to suitable scale the differentials of the E-curve. A third curve will thus be obtained which may be called the discharge curve. This curve will give at once the value of d_m for any value of t_m.

By calling $t_m = t_D$, a quick first approximation of the value d_m can thus be made.

It follows from the preceding demonstrations that for every mass

curve of surface run-off, on the hypotheses as made, there is a definite inter-relation between the values of S_m, t_m, and d_m. This inter-relation can be determined from the diagram, and can be tabulated.

Ordinarily, the fact most readily ascertainable for any district is the value of t_D, which (in the case of urban storm-water flow problems) may be accepted as a first approximation of the value of t_m. In tabulating the information obtained by diagram it will, therefore, be convenient to note regular time intervals in the first column.

The values noted in Columns 2, 3, and 4 of Table 7 are based on the limiting rain curve for San Francisco, as determined by the formula,

$$I = \frac{3.68}{\frac{2\,t}{t+60} + t^{0.4}}$$

and are applicable to areas with impervious surface throughout.

The corresponding columns in Table 8 contain values based on the same limiting rain curve with reductions due to water absorption by soil when only 25% of the surface is impervious. (For the reduction factors used see Table 6.)

TABLE 7.—Storm-Water Flow in Its Relation to the Duration of the Critical Time Period, and to the Augmentation of the Water Quantity Temporarily Stored in the Conduits.

For areas with impervious surface throughout. For San Francisco.

Based on $I = \frac{3.68}{\frac{2\,t}{t+60} + t^{0.4}}$.

Duration of critical period, t_m, in minutes.	Augmentation of storage in the conduits, $C_T - C_B$, in cubic feet per acre.	By Diagram.		Discharge, by method used in 1899, calling $t_m = t_D$, in second-feet per acre.
		Augmentation of storage on the ground and in the conduits, S, in cubic feet per acre.	Discharge, d_m, in second-feet per acre.	
5	5	200	1.25	1.34
10	270	300	0.93	0.99
15	365	385	0.78	0.82
20	440	450	0.69	0.71
30	545	555	0.57	0.60
40	635	640	0.51	0.54
50	725	730	0.46	0.49
60	800	800	0.43	0.45
70	870	870	0.40	0.43
80	930	930	0.38	0.41
90	985	985	0.36	0.38
120	1 120	1 120	0.32	0.34

TABLE 8.—STORM-WATER FLOW, IN ITS RELATION TO THE DURATION OF THE CRITICAL PERIOD AND TO THE AUGMENTATION DURING SUCH PERIOD OF THE WATER QUANTITY TEMPORARILY STORED IN THE CONDUITS.

For areas with 25% of the surface impervious. For San Francisco.

$$\text{Based on } I = \frac{3.68}{\frac{2\,t}{t+60} + t^{0.4}}.$$

Duration of critical period, t_m, in minutes.	Augmentation of the storage in the conduits, $C_T - C_B$, in cubic feet per acre.	By Diagram.		Discharge, by method used in 1899, calling $t_m = t_D$, in second-feet per acre.
		Augmentation of storage on the ground and in the conduits, S, in cubic feet per acre.	Discharge, d_m, in second-feet per acre.	
5	5	140	0.89	0.63
10	170	190	0.60	0.46
15	205	220	0.45	0.39
20	230	240	0.37	0.34
30	265	270	0.28	0.28
40	290	295	0.24	0.25
50	320	320	0.20	0.23
60	345	345	0.19	0.21
90	375	375	0.14	0.18
120	415	415	0.12	0.16

Referring again to Fig. 7, it remains to be shown that no rain condition preceding the time, B, can be assumed which will give the line, $E'\,D'$, greater inclination than resulting from that which has already been declared to be the most unfavorable.

Assume the extreme case of no rain preceding the time, B. In this case there will be no storm-water in the conduits and no water on the ground at the time, B. Consequently, $C_B = 0$, and $G_B = 0$.

From Equation 11, $S = C_T + G_T$;

but, as has already been shown, the maximum value of C_B for extreme rain intensity preceding the time, B, is about one-half of C_T; therefore, approximately, for this condition,

$$C_T = 2\,(C_T - C_B),$$

where C_B represents volumetric water contents of the conduits due to maximum rainfall conditions prior to the time, B.

For the special case shown in the diagram,

for $t = 50$, $C_T - C_B = 730$,
therefore $C_T = 1\,460$.
From Table 5, $G_T = 75$
consequently $S = 1\,535$.

If the critical time were the same for the condition of no rain preceding the time, B, as for the condition of heavy rain preceding B, then it would be proper to scale this value of S downward from T in Fig. 9, and it would at once be apparent, because the inclination of E'' D'' is less than the inclination of E' D', that the most unfavorable assumption is that of heavy rain preceding the critical period. But the durations of the critical periods are not the same in the two cases. On the same hypothesis on which Equation 5 is based, the critical period for the case of no preceding rain is of nearly twice the duration as for the case of heavy preceding rain. For the special values above noted, one-third of the critical period would be about 30 min., and the corresponding value of d'_m would be the inclination of the broken line (Fig. 9) rising from the 30-min. point of the base line.

This, also, is less than that resulting from the originally assumed unfavorable conditions. The same result can be shown for any other value of t and for any other rain intensities preceding the time, B, less than those originally assumed as the least favorable.

The conclusion relating to the most unfavorable distribution of rain heretofore announced appears therefore to be correct. The rain rates that should be assumed as prevailing before the beginning of the critical period are the greatest rates that are possible, the rain being such that it falls with gradually increasing intensity until the time, B.

The comparison of the results by the new method of computation with those obtained by the method of 1899, as shown in Table 7, indicate reasonably close agreement for areas with impervious surface.

The comparison in Table 8 for areas which are only in small part impervious indicates a greater range of unit values for the new method than result from the older method of calculation. The method of 1899 probably gives too small values for small areas which are only partly impervious.

It may be repeated that water quantities in the conduits at the times, B and T, can ordinarily be calculated with a sufficient degree of approximation. At the time, T, the main conduit is flowing at capacity at the point for which storm-water flow is to be estimated, and the inlets on all the conduit branches are at the same time receiving water at the run-off rate which corresponds to the last few minutes of

the critical time period. It is possible, therefore, to determine to what proportion of their capacity the conduits are charged at their inlet ends. With this information for the upper ends of conduits, and the knowledge that the conduits are full near the point under consideration, a close approximation of the water stage at the time, T, is possible, and C_T may be calculated.

By a similar line of reasoning, the stage of water in the conduits can be determined for the time, B, and the value of C_B can also be calculated.

But for first approximation purposes it will be well to establish the values of K for increasing areas. Then call the ratio of the water stored in the conduits at the time, B, to the water stored at the time, T, equal to the ratio of the mean run-off during the t_D minutes preceding B to the mean run-off in the t_D minutes after B. For San Francisco this ratio, as already stated, may be called as one to two.

$$\text{That is,} \qquad C_B = \frac{C_T}{2};$$

$$\text{but} \qquad C_T = K\,C,$$

$$\text{therefore} \qquad C_B = \frac{K\,C}{2},$$

$$\text{and } C_T - C_B = \frac{K\,C}{2}.$$

To the value of $C_T - C_B$ thus or otherwise determined add the value of $G_T - G_{T+i}$, as found in Column 7 of Table 5 for $i = 5$. The sum will be the value of S to be used, as already explained.

When this method of determining storm-water flow is applied to a conduit system already constructed, it may happen that, owing to excessive or deficient storage capacity in the conduits, there will be wide divergence in the values of t_m and t_D.

The value of t_m may be considerably less than t_D if the conduits are too small, if they are surcharged at critical times; and it may be materially larger than t_D if along the line of the conduits there is unusual reservoir space to be filled before the maximum stage is attained. It will be nearly twice as great as t_D for the special case of no rain preceding the critical period.

The graphical method of determining the storm-water flow is applicable to any limiting mass curve of surface run-off. It is applicable therefore to any such curve obtained by applying a reduc-

tion factor to the run-off rates from areas with impervious surfaces.

But when the problem is presented of finding the storm-water flow for districts having surfaces which are not impervious throughout, it will not be necessary to construct new curves. The original curve for impervious conditions throughout may be used to solve any such problem.

When the value of $S = C_T - C_B + G_T - G_{T+t}$ has been determined for an area of any surface character, divide this value by the factor, a (to be found in Table 6), and proceed with the modified value of S thus obtained ($S' = \frac{S}{a}$), as though the area had an impervious surface throughout. There will thus be obtained a modified value of d_T, which may be called d'_T, such that

$$d'_T = \frac{d_T}{a},$$

$$\text{therefore,} \qquad d_T = a\, d'_T.$$

That is to say, the value thus obtained from the curves or from the tables is to be multiplied by the same reduction factor before used, to give the value of d_T.

New Formula and Discharge Tables.—A study of the limiting rain curve of 1899 for San Francisco has led to the adoption of a formula, as already stated, which, it is believed, will be found generally applicable for the determination of rain intensities for all periods of time that come under consideration in urban run-off problems.

In its general form the formula may be written

$$I = \frac{b}{\frac{2\,t}{t+60} + t^{0.5}} \qquad \text{.....(21)}$$

Here b is a constant for any locality, and t is the duration of the rain, in minutes.

This formula gives the value of I, in inches per hour, or, which is the same thing, in cubic feet per second per acre.

For San Francisco, $b = 3.68$ will give values closely agreeing with those determined in 1899. (See Table 3.)

For Atlantic Slope conditions, ordinarily, $b = 8$ to 12.

Wherever the maximum rain intensity for 1 hour is known, the value of b can be determined. It will generally be well, however, provided data are adequate, to base the value of b upon determinations from the maximum rain intensities for 10 min., 30 min., and 1 hour. In most cases the use of an average value from these three determinations should satisfy every requirement.

The rain intensities resulting from $b = 3.68$, 4, 8, and 12, in this formula, have already been noted in Table 3.

Whether the expression recommended, or any other expression in the general form

$$I = c\, f\,(t) \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (22)$$

be used in estimating the maximum rain intensity during the time, t, the fact is noteworthy that for all possible values of c or of b in the other case, the corresponding ordinates of the resulting mass curves will be proportional to the values of c or b that have been used.

This fact makes it possible to establish by a single diagram (for impervious surface conditions), base values of storage augmentation and of storm-water flow, being for the special case of $b = 1$ or of $c = 1$.

From such base values the storage and discharge for other values of b or of c, and also for any condition of the surface, can then be found by multiplication with the factors, a, b or a c, as the case may be.

Table 9 contains the base values of S_m, d_m, and $(C_T - C_B)$, that is $(S_m)_{\substack{a=1\\b=1}}$, $(d_m)_{\substack{a=1\\b=1}}$, and $(C_T - C_B)_{\substack{a=1\\b=1}}$, for the type of rain curve herein recommended, as determined by diagram.

The values in the last column of Table 9 conform closely to a curve herein recommended, as determined by diagram.

$$y = \frac{0.71}{\frac{2\,t_m}{t_m + 60} + t_m{}^{0.4}} \ldots\ldots\ldots\ldots\ldots\ldots\ldots (23)$$

This is the value of d_m for the special case in which $a = 1$ and $b = 1$. For any value of a and of b, therefore,

$$d_m = \frac{0.71\, a\, b}{\frac{2\,t_m}{t_m + 60} + t_m{}^{0.4}} = 0.71\, a\, I \ldots\ldots\ldots\ldots\ldots\ldots (24)$$

As it frequently will be desirable to compute the discharge directly from the maximum rainfall in 1 hour, the value of b may be found in terms of R and substituted.

For $t = 60$, there will be $I = R$.

$$I = \frac{b}{\frac{2\,t}{t + 60} + t^{0.4}} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (21)$$

$$R = \frac{b}{6.14} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (25)$$

$$b = 6.14\, R \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (26)$$

Therefore, from Equation 24,

$$d_m = \frac{4.36\ a\ R}{\frac{2\ t_m}{t_m + 60} + {t_m}^{0.4}} = 4.36\ a\ R \left(\frac{I}{b}\right) \quad \ldots\ldots\ldots\ldots (27)$$

TABLE 9.—BASE VALUES OF d_m AND $C_T - C_B$; WHEN $a = 1$ AND $b = 1$;

IN THE FORMULA FOR SURFACE RUN-OFF:

$$r = a\ I = \frac{a\ b}{\frac{2\ t}{t + 60} + t^{0.4}}$$

All values determined by diagram.

(Subject to correction.)

Duration of the critical period, t_m, in minutes.	Total storage increase, (S_m) $a = 1$ $b = 1$, in cubic feet per acre.	Increase of storage in the conduits, $(C_T - C_B)$ $a = 1$ $b = 1$, in cubic feet per acre.	Discharge, (d_m) $a = 1$ $b = 1$, in second-feet per acre.
5	54	0	0.340
10	82	73	0.252
15	105	100	0.212
20	122	120	0.187
30	151	150	0.156
40	174	170	0.139
50	198	197	0.125
60	218	218	0.116
90	268	268	0.097
120	305	305	0.087

In the practical application of this formula, a should be considered constant for each area, as already explained. Its value is directly dependent upon the rate of water absorption by exposed soil surfaces during heavy rains. The value of a for any value of b and any value of t, therefore, can be determined by experiment. For surfaces impervious throughout, the value of a is unity.

Some other values of a preliminarily suggested for use in this formula are to be found in Table 6.

The foregoing discharge formula will be directly applicable to the determination of the maximum discharge when, for any district under consideration, the value of t which will make d_T a maximum is known.

As a first approximation, without great error, t_m may be called equal to t_D. Consequently, a first approximation of d_T by the formula is possible whenever the time can be estimated that it will

take water to flow from the most remote portions of a district to the point at which discharge is being estimated.

It can readily be seen, when

$$I = \frac{b}{\frac{2\,t}{t+60} + t^{0.4}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (21)$$

$$\text{that } (G_T - G_{T+i}) = a\,b\,[G_T - G_{T+i}]_{\substack{a=1\\b=1}} \ldots\ldots\ldots (28)$$

$$\text{and that } \quad (C_T - C_B) = a\,b\,[C_T - C_B]_{\substack{a=1\\b=1}} \ldots\ldots\ldots\ldots (29)$$

consequently, also,

$$S = a\,b\,(S)_{\substack{a=1\\b=1}} \ldots\ldots\ldots\ldots\ldots\ldots\ldots (30)$$

From the values given for $(S_m)_{\substack{a=1\\b=1}}$ in Table 9, for the special case of $i = 5$, the relation between t_m and $(S_m)_{\substack{a=1\\b=1}}$ can be reduced to formula. It will be found that, with a sufficient degree of accuracy for all values of t_m which are likely to arise in connection with urban problems,

$$(S_m)_{\substack{a=1\\b=1}} = 27.5\,\sqrt{t_m} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (31)$$

herefore

$$S_m = 27.5\,a\,b\,\sqrt{t_m} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (32)$$

and

$$t_m = \left(\frac{S_m}{27.5\,a\,b}\right)^2 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (33)$$

From Equations 11 and 32,

$$C_T - C_B = 27.5\,a\,b\,\sqrt{t_m} - (G_T - G_{T+i}) \quad \ldots\ldots\ldots\ldots (34)$$

$$C_T - C_B = 27.5\,a\,b\,\sqrt{t_m} - a\,b\,(G_T - G_{T+i})_{\substack{a=1\\b=1}} \quad \ldots\ldots (35)$$

and when $i = 5$,

$$C_T - C_B = a\,b\left[27.5\,\sqrt{t_m} - (G_T - G_{T+5})_{\substack{a=1\\b=1}}\right] \ldots\ldots\ldots (36)$$

It has already been shown that

$$b = 6.14\,R \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (26)$$

Consequently, from Equation 32,

$$S_m = 169\,a\,R\,\sqrt{t_m} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (37)$$

and from Equation 36,

$$C_T - C_B = a\,R\left[169\,\sqrt{t_m} - 6.14\,(G_T - G_{T+5})_{\substack{a=1\\b=1}}\right] \ldots\ldots (38)$$

Likewise, from Equation 37,

$$t_m = \left(\frac{S_m}{169\,a\,R}\right)^2 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (39)$$

and, convenient for purposes of preliminary approximation, from Equations 27 and 37,

$$d_m = \frac{S_m}{38.8\sqrt{t_m}}\left(\frac{1}{\frac{2\,t_m}{t_m+60}+t_m^{0.4}}\right) \quad \ldots\ldots\ldots\ldots (40)$$

TABLE 10.—STORM-WATER FLOW.

FOR $b = 4$, IN THE FORMULA, $I = \dfrac{b}{\frac{2\,t}{t+60}+t^{0.4}}$

All values in this table are based on

$$d_m = \frac{0.71\,a\,b}{\frac{2\,t_m}{t_m+60}+t_m^{0.4}} = \frac{4.36\,a\,R}{\frac{2\,t_m}{t_m+60}+t^{0.4}}$$

$$S_m = 27.5\sqrt{t_m}$$

$$\text{and } C_T - C_B = S_m - (G_T - G_{T+5})$$

$$b = 4 \text{ and } R = 0.65.$$

Duration of critical period, t_m, in minutes.	ALL IMPERVIOUS.		75% IMPERVIOUS.		50% IMPERVIOUS.		25% IMPERVIOUS.		NONE IMPERVIOUS.	
	$C_T - C_B$ cubic feet per acre.	d_m second-feet per acre	$C_T - C_B$ cubic feet per acre.	d_m second-feet per acre.	$C_T - C_B$ cubic feet per acre.	d_m second-feet per acre.	$C_T - C_B$ cubic feet per acre.	d_m second-feet per acre.	$C_T - C_B$ cubic feet per acre.	d_m second-feet per acre.
5	35	1.38	35	1.26	30	1.13	30	1.01	25	0.88
10	310	1.02	280	0.91	250	0.79	210	0.67	165	0.55
15	400	0.85	350	0.74	295	0.63	240	0.51	190	0.40
20	480	0.74	410	0.63	345	0.53	255	0.42	205	0.32
30	600	0.62	500	0.53	410	0.43	315	0.33	220	0.23
40	690	0.55	580	0.46	460	0.37	350	0.28	235	0.185
50	780	0.50	650	0.42	510	0.33	380	0.25	240	0.155
60	850	0.46	700	0.38	550	0.30	400	0.215	245	0.135
90	1 050	0.39	860	0.32	660	0.245	460	0.170	260	0.098
120	1 210	0.35	980	0.28	740	0.215	510	0.145	275	0.080

When, therefore, the increase of water storage during a critical time period can be determined, the duration of this period can be found from Equations 33 or 39, and d_m can be estimated by formula, Equations 24 or 29; or the value of d_m can be taken from Tables 10, 11, and 12, or from the general table, Table 13. Or, the value of d_m may be estimated directly from the value of S_m and an approximate value of t_m by the use of Equation 40. In this last case the estimated discharge is to be treated as an approximation until the value of t_m has been verified.

TABLE 11.—Storm-Water Flow.

For $b = 8$, in the Formula, $I = \dfrac{b}{\dfrac{2\,t}{t+60} + t^{0.4}}$

All values in this table are based on

$$d_m = \frac{0.71\,a\,b}{\dfrac{2\,t_m}{t_m+60} + t_m^{0.4}} = \frac{4.36\,a\,R}{\dfrac{2\,t_m}{t_m+60} + t^{0.4}}$$

$$S_m = 27.5\,a\,b\,\sqrt{t_m}$$

$$\text{and } C_T - C_B = S_m - (G - G_{T+5})$$

$$b = 8 \text{ and } R = 1.30.$$

Duration of critical period, t_m, in minutes.	All impervious.		75% impervious.		50% impervious.		25% impervious.		None impervious.	
	$C_T - C_B$ cubic feet per acre.	d_m second-feet per acre.	$C_T - C_B$ cubic feet per acre.	d_m second-feet per acre.	$C_T - C_B$ cubic feet per acre.	d_m second-feet per acre.	$C_T - C_B$ cubic feet per acre.	d_m second-feet per acre.	$C_T - C_B$ cubic feet per acre.	d_m second-feet per acre.
5	70	2.77	70	2.65	65	2.52	65	2.40	60	2.27
10	620	2.03	590	1.91	550	1.79	520	1.68	480	1.56
15	810	1.70	760	1.58	710	1.48	660	1.37	600	1.26
20	960	1.49	900	1.38	830	1.28	760	1.18	690	1.07
30	1 190	1.24	1 100	1.14	1 010	1.04	920	0.94	820	0.86
40	1 380	1.10	1 270	1.01	1 150	0.91	1 040	0.82	920	0.74
50	1 550	1.00	1 440	0.91	1 330	0.82	1 170	0.74	1 010	0.65
60	1 710	0.93	1 560	0.85	1 410	0.76	1 260	0.68	1 100	0.60
90	2 090	0.78	1 900	0.71	1 700	0.64	1 510	0.56	1 310	0.49
120	2 420	0.70	2 190	0.64	1 960	0.57	1 730	0.50	1 500	0.43

Tables 10, 11, 12, and 13 have been prepared by the use of the foregoing formulas, all based on

$$I = \frac{b}{\dfrac{2\,t}{t+60} + t^{0.4}}$$

$$i = 5$$

and an absorption of water by the soil at the rate of about $\frac{3}{4}$ in. per hour at the beginning of a critical period, decreasing gradually to a rate of $\frac{1}{3}$ in. per hour at the end of 1 hour.

In the case of conduits already constructed, neither the value of $C_T - C_B$, nor the value of d_m, taken from the tables for $t_m = t_D$, should be considered final.

To facilitate the use of the formula and of the reduction factors, a, Table 13 has been prepared. In this table the value of

$$\frac{0.71}{\dfrac{2\,t_m}{t_m+60} + t_m^{0.4}}$$

appears in Column 3, and the value of

$$\left[27.5\sqrt{t_m} - (G_T - G_{T+5})\right]_{\substack{a=1\\b=1}}$$

in Column 2. The value of $a\ b$ is given in Columns 4 to 18.

To find the discharge, by this table, multiply the value found in Column 3 by the appropriate factor taken from Columns 4 to 18; and to find the increase of the water stored in the conduits, multiply the values found in Column 2 by the same factor.

The table, as already stated, is based on the assumption that $i = 5$; and that, during the time, t_m, the absorption of water by exposed soil surface is at rates decreasing gradually from about $\frac{2}{3}$ in. per hour to $\frac{1}{3}$ in. per hour at the end of 1 hour.

TABLE 12.—STORM-WATER FLOW.

FOR $b = 12$, IN THE FORMULA, $I = \dfrac{b}{\dfrac{2\ t}{t+60} + t^{0.4}}$

All values in this table are based on

$$d_m = \frac{0.71\ a\ b}{\dfrac{2\ t_m}{t_m+60} + t_m^{\,0.4}} = \frac{4.36\ a\ R}{\dfrac{2\ t_m}{t_m+60} + t_m^{\,0.4}}$$

$$S_m = 27.5\ a\ b\ \sqrt{t_m}$$

and $C_T - C_B = S_m - (G_T - G_{T+5})$

$$b = 12 \text{ and } R = 1.95.$$

Duration of critical period, t_m, in minutes.	ALL IMPERVIOUS.		75% IMPERVIOUS.		50% IMPERVIOUS.		25% IMPERVIOUS.		NONE IMPERVIOUS.	
	$C_T - C_B$ cubic feet per acre.	d_m second-feet per acre.	$C_T - C_B$ cubic feet per acre.	d_m second-feet per acre.	$C_T - C_B$ cubic feet per acre.	d_m second-feet per acre.	$C_T - C_B$ cubic feet per acre.	d_m second-feet per acre.	$C_T - C_B$ cubic feet per acre.	d_m second-feet per acre.
5	100	4.15	100	4.03	95	3.90	90	3.78	90	3.65
10	930	3.05	900	2.95	860	2.84	820	2.72	780	2.60
15	1 210	2.54	1 160	2.44	1 110	2.34	1 060	2.21	1 010	2.11
20	1 440	2.23	1 380	2.14	1 310	2.03	1 240	1.92	1 170	1.80
30	1 790	1.86	1 700	1.76	1 610	1.68	1 520	1.58	1 420	1.47
40	2 070	1.66	1 960	1.57	1 840	1.48	1 730	1.39	1 610	1.29
50	2 330	1.50	2 200	1.41	2 060	1.32	1 930	1.25	1 790	1.15
60	2 560	1.39	2 410	1.31	2 260	1.22	2 110	1.14	1 950	1.06
90	3 140	1.18	2 950	1.11	2 750	1.04	2 550	0.95	2 350	0.88
120	3 620	1.04	3 390	0.98	3 150	0.90	2 910	0.83	2 670	0.77

TABLE 13.—VALUES OF $a\,b$,

$$\text{WHEN } I = \frac{b}{\frac{2\,t}{t+60} + t^{0.4}} \quad \text{AND } d_m = a\,b\,\frac{0.71}{\frac{2\,t_m}{t_m+60} + t_m{}^{0.4}}$$

$$S_m = a\,b\,(27.5\,\sqrt{t_m}) \text{ and } C_T - C_B = a\,b\,[C_T - C_B]_{\substack{a=1\\ b=1}}.$$

Duration of critical period, t_m, in minutes.	Base Values, for $b=1$ and $a=1$.		Values of $a\,b$:									
	$C_T - C_B$ cubic feet per acre.	d_m second-feet per acre.	For $b=4$ and $R=0.65$.				For $b=8$ and $R=1.30$.				For $b=12$ and $R=1.95$.			
			All impervious.	75% impervious.	50% impervious.	25% impervious.	None impervious.	All impervious.	75% impervious.	50% impervious.	25% impervious.	None impervious.	All impervious.	75% impervious.	50% impervious.	25% impervious.	None impervious.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)
5	8	0.347	4.00	3.65	3.30	2.90	2.60	8.00	7.70	7.30	6.95	6.55	12.00	11.60	11.30	10.90	10.60
10	78	0.254	4.00	3.55	3.10	2.65	2.15	8.00	7.50	7.05	6.65	6.15	12.00	11.55	11.15	10.70	10.20
15	107	0.212	4.00	3.50	2.95	2.40	1.90	8.00	7.45	6.95	6.50	5.90	12.00	11.50	10.95	10.45	9.95
20	125	0.186	4.00	3.45	2.90	2.25	1.70	8.00	7.40	6.90	6.30	5.75	12.00	11.45	10.90	10.30	9.70
30	150	0.156	4.00	3.40	2.75	2.10	1.50	8.00	7.35	6.75	6.10	5.50	12.00	11.40	10.80	10.20	9.50
40	170	0.138	4.00	3.35	2.70	2.00	1.35	8.00	7.35	6.65	6.00	5.30	12.00	11.35	10.70	10.10	9.35
50	195	0.125	4.00	3.30	2.65	1.95	1.25	8.00	7.30	6.60	5.90	5.20	12.00	11.30	10.60	9.95	9.20
60	215	0.116	4.00	3.30	2.60	1.90	1.15	8.00	7.30	6.55	5.80	5.10	12.00	11.30	10.55	9.80	9.10
90	260	0.098	4.00	3.25	2.50	1.75	1.00	8.00	7.25	6.50	5.75	5.00	12.00	11.30	10.50	9.70	9.00
120	300	0.087	4.00	3.25	2.50	1.70	0.90	8.00	7.25	6.50	5.75	4.95	12.00	11.30	10.50	9.60	8.90

The type of the formula for rain intensity, as here recommended, is the direct outcome of an attempt to make the resulting values follow closely the San Francisco limiting rain curve for all values of t up to 24 hours.

The parabola, as a limiting curve, was tried, but, for higher values of t, departed too far from the curve of 1899 to be considered entirely satisfactory.

When all time periods that come into consideration are less than 2 hours, as will be the case in nearly all urban problems, a parabola can frequently be found which will closely approximate the limiting rain curve, and may be used in its stead.

In such case let the equation of the limiting rain curve be

$$q = e\sqrt{t} \text{ (total rainfall in inches)} \quad \ldots\ldots (41)$$

The equation of the surface run-off curve will then be

$$q' = a\, e\sqrt{t} \quad \ldots\ldots (42)$$

The value of a, as has already been explained, is dependent upon the amount of impervious surface in any area, and varies, too, with the duration of the critical period. For impervious areas, a is unity.

For $t = 60$, the value of q will be the maximum rainfall in 1 hour, therefore, from Equation 41,

$$R = e\sqrt{60} \quad \ldots\ldots (43)$$

$$e = \frac{R}{7.746} \quad \ldots\ldots (44)$$

$$e = 0.129\, R \quad \ldots\ldots (45)$$

$$\text{therefore } q' = 0.129\, a\, R\sqrt{t} \quad \ldots\ldots (46)$$

The maximum intensity of the rainfall for any number of minutes can be found from

$$I = \frac{60\, q}{t} = \frac{60\, e}{\sqrt{t}} \quad \ldots\ldots (47)$$

$$\text{and } I = \frac{7.75\, R}{\sqrt{t}} \quad \ldots\ldots (48)$$

By reference to Fig. 10 it can be seen that for a curve the equation for which is

$$q' = a\, e\sqrt{t} \quad \ldots\ldots (42)$$

the equation of the line, $B'\ T$, can be written

$$y = \frac{0.414\, a\, e}{\sqrt{t}}\, x + 0.586\, a\, e\sqrt{t} \quad \ldots\ldots (49)$$

For $x = \frac{2\, t}{3}$, this equation will give the ordinate of the point, E.

The locus of the point, E, will be expressed by

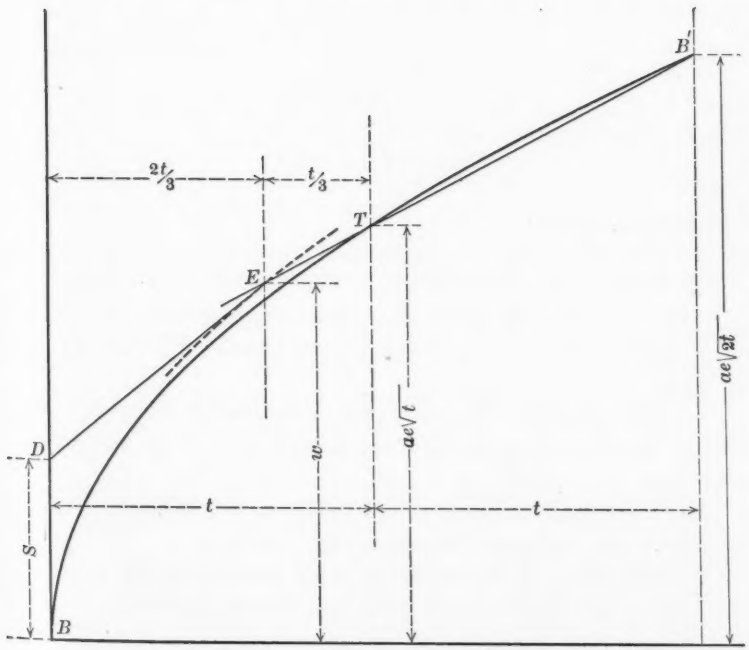
$$w = \frac{0.828\ a\ e\ t}{3\sqrt{t}} + 0.586\ a\ e\sqrt{t} \quad \ldots\ldots\ldots\ldots\ldots\ldots (50)$$

$$w = 0.862\ a\ e\sqrt{t} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (51)$$

$$\text{or } w = 1.056\ a\ e\sqrt{x} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (52)$$

Therefore the tangent to the E-curve at the point, E,

$$\frac{d\,w}{d\,x} = \frac{0.528\ a\ e}{\sqrt{x}} = \frac{0.647\ a\ e}{\sqrt{t}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (53)$$



WHEN THE LIMITING RAIN CURVE IS A PARABOLA

FIG. 10.

The value of $\frac{d\,w}{d\,x}$ as thus found will be inches in depth in 1 min. Therefore d_m, in inches in 1 hour, or in second-feet per acre,

$$d_m = 60\ \frac{0.647\ a\ e}{\sqrt{t}} = \frac{38.8\ a\ e}{\sqrt{t_m}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (54)$$

And, by substitution,

$$d_m = \frac{5.00\ a\ R}{\sqrt{t_m}} \text{ cu. ft. per sec. per acre} \quad \ldots\ldots\ldots\ldots\ldots\ldots (55)$$

The value of S_m, expressed in inches of depth, will be

$$w - \frac{2\ t_m}{3}\left(\frac{0.647\ a\ e}{\sqrt{t_m}}\right)$$

from which it follows:

$$S_m = 3\ 630\ [0.862\ a\ e\ \sqrt{t_m} - 0.431\ a\ e\ \sqrt{t}\ ] \quad \ldots\ldots (56)$$

$$S_m = 1\ 565\ a\ e\ \sqrt{t_m} \text{ cu. ft. per acre} \quad \ldots\ldots (57)$$

From Equation 57, $$t_m = \left(\frac{S_m}{1\ 565\ a\ e}\right)^2 \quad \ldots\ldots (58)$$

By substitution from Equation 45,

$$t_m = \left(\frac{S_m}{202\ a\ R}\right)^2 \quad \ldots\ldots (59)$$

and $$S_m = 202\ a\ R\ \sqrt{t_m} \quad \ldots\ldots (60)$$

Finally, from Equations 54 and 58,

$$d_m = \frac{60\ 700\ a\ e^2}{S_m} \quad \ldots\ldots (61)$$

and from Equations 55 and 60,

$$d_m = \frac{S_m}{40.4\ t_m} \quad \ldots\ldots (62)$$

The value of a, for use in the foregoing equations, may be taken from Table 6, for the soil absorption conditions on which that table is based.

For San Francisco, the value of e to be used in these equations is 0.0775, the value of R, the greatest possible rainfall in 1 hour, being 0.60 in.

For the rainfall conditions prevailing on the Atlantic Slope, the value of e will ordinarily lie between 0.15 and 0.25.

In order that the importance of using good judgment in the construction of the limiting rain curve may become apparent, Tables 14 and 15 have been prepared.

In Table 14 corresponding values of S_m and d_m are given for the various values of t_m, estimated in the one case by the use of formulas based on limiting rain curves of the San Francisco type, and in the other by the use of formulas based on the parabola type of curve. This table is of general application.

In Table 15 the comparison is made between corresponding values of t_m and d_m for selected values of S_m; but in this case a specific value of R was assumed: $R = 0.60$. The figures on the same lines are not those that would result from an adaptation of curves of the two

types to the rain records, because properly selected curves may not have coincident values of R.

For the San Francisco type of curves:

$$I = \frac{b}{\frac{2\,t}{t+60} + t^{0.4}}$$

$$d_m = \frac{4.36\,a\,R}{\frac{2\,t_m}{t_m+60} + t_m{}^{0.4}}$$

and $$S_m = 169\,a\,R\,\sqrt{t_m}$$

For the parabola:

$$I = \frac{60\,e}{\sqrt{t}}$$

$$d_m = \frac{5\,a\,R}{\sqrt{t_m}} = \frac{S}{40.4\,t}$$

$$S_m = 202\,a\,R\,\sqrt{t_m}$$

TABLE 14.—COMPARATIVE VALUES OF STORM-WATER FLOW ESTIMATED FOR TWO TYPES OF THE LIMITING RAIN CURVE.

For impervious areas throughout.

$a = 1$

Duration of the critical period, t_m, in minutes.	FOR A LIMITING RAIN CURVE OF THE SAN FRANCISCO TYPE.		WHEN THE LIMITING RAIN CURVE IS A PARABOLA.	
	S_m, cubic feet per acre.	d_m, second-feet per acre.	S_m, cubic feet per acre.	d_m, second-feet per acre.
5	378 R	2.13 R	452 R	2.24 R
10	535 R	1.56 R	640 R	1.58 R
15	655 R	1.30 R	780 R	1.29 R
20	755 R	1.14 R	905 R	1.12 R
30	930 R	0.96 R	1 010 R	0.91 R
40	1 070 R	0.85 R	1 280 R	0.79 R
50	1 200 R	0.77 R	1 430 R	0.71 R
60	1 310 R	0.71 R	1 570 R	0.65 R
90	1 600 R	0.60 R	1 920 R	0.53 R
120	1 850 R	0.54 R	2 220 R	0.46 R

Application of the New Method to Special Cases.—Should it be desired to know the extent to which a branch conduit contributes to the discharge of a main conduit at the time, T, two procedures are possible. The discharge estimate may be made for the main conduit for points just above and just below the entrance of the sub-main. The difference between the two results will be the required discharge of the sub-main.

TABLE 15.—COMPARATIVE VALUES OF STORM-WATER FLOW ESTIMATED FOR TWO TYPES OF THE LIMITING RAIN CURVE.

For $R = 0.60$.

For impervious areas throughout.

$a = 1$, and $R = 0.60$.

Increase of storage, S_m, in cubic feet per acre.	For a Limiting Rain Curve of the San Francisco Type.		When the Limiting Rain Curve is a Parabola.	
	t_m, minutes.	d_m, second-feet per acre.	t_m, minutes.	d_m, second-feet per acre.
200	3.89	1.43	2.72	1.82
250	6.08	1.17	4.25	1.46
300	8.75	0.99	6.11	1.21
400	15.6	0.77	10.9	0.91
500	24.2	0.64	16.9	0.73
600	35.0	0.54	24.4	0.61
700	47.5	0.47	33.4	0.52
800	62.0	0.42	43.6	0.45
900	78 6	0.38	55.0	0.40
1 000	97.3	0.35	68.1	0.36

Or, as may frequently be convenient, the sub-main is considered separately. In this event it must be remembered that the mass curve of surface run-off, which is of most unfavorable shape for the main district, is as shown in Fig. 8. During the first part of the critical period, the rainfall will have its greatest intensity. The run-off produced by the rain of greatest intensity will have time to escape from the sub-district (supposed to be relatively small) before the main conduit flows at capacity. Therefore, in applying the graphical method, only that part of the run-off curve is to be used which lies nearest to T. But, for decreasing intensities of rain, in a small district, there will probably be less water in the conduits at the time, T, than t_D minutes before T (t_D now applying to the sub-district). Consequently, instead of an augmentation of water quantity stored in the conduits there will be a depletion of this storage. The value of $C_T - C_B$ or of S, as the case may be, will be negative, and must be platted upward from the curve at T, instead of downward.

In the supposed case, Fig. 8, the sub-district time is 20 min. and the required discharge of the sub-main is represented by the inclination of the line, $E_1 D_1$.

It may be of interest to note that the described method of estimating flow at any point of a water conduit when the rates are known at which

water is supplied to it by its feeders, is applicable to rivers and canals* as well as to covered conduits, and also to conduits operating in part under pressure.

A closed conduit serving throughout under pressure is an extreme case. There can then be no augmentation of the quantity of water stored in the conduit. The effect of an increase or decrease in the rate of supply will be instantaneous from end to end of the conduit. The time, t, will become zero. The points, B, T, and E on the curve will all be at the origin of the co-ordinates, and the maximum rate of flow at all points of the conduit will be equal to the maximum rate at which water is supplied, because, for $t = 0$, the E-curve is tangent to the supply curve at the point, B.

At the other extreme is the case of flow through large reservoirs or basins. In this case, the storage of water is large, the time, t, is relatively long, and the maximum rate of outflow is correspondingly small.

In applying the method of estimating required storm-water capacity, as described, it is well to remember that a proper determination of the value of b eliminates the necessity for introducing into the calculation any further allowance for safety margin except such as may be covered by a selection of a value for the factor a. As a is unity for areas with impervious surfaces throughout and as the value of a is otherwise definitely determinable, it will be seen that the room for error is small.

General Deductions.—It appears from the foregoing:

1.—That the method as described of estimating storm-water flow is applicable to districts of any shape, size, or surface condition.

2.—That if a conduit affords no reservoir space, the storm-water flow at all points thereof would be equal to the rate of inflow at the storm-water inlets, and the effect of an increase or decrease of this rate would be immediately manifest at all points of the conduit. (This is the case of a conduit under pressure.)

3.—That the longer the time required for storm-water concentration, the larger will be the unit storage capacity of the conduits in any district, or *vice versa,* and the greater the influence of the conduit storage upon the rate of flow.

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* This is substantially the method of estimating river flow described by the writer in a recent discussion of a paper on a California flood. *Transactions*, Am. Soc. C. E., Vol. LXI, p. 332.

4.—That an outfall conduit, receiving no further accessions, below a certain point:

a.—If occasionally or always submerged must be given uniform capacity from that point to the point of outfall;

b.—If not flowing under pressure, and if it have a free outfall not obstructed by back-water, its maximum discharge will decrease somewhat with increasing distance below that point.

If the shape of the limiting rain curve has been properly determined from adequate rainfall observations, the secondary curves derived therefrom should be smooth curves. The value of discharge determined therefrom will, in other words, follow a definite law of change.

It is obvious that the graphical method of determining discharge may be applied to any rain storm for which a mass curve of the rainfall can be platted. The proper location of the points, B and T, on the resulting surface run-off curve will be found by trial.

The method of estimating storm-water flow as here described commends itself for use whenever the limiting rain curve is known:

First.—Because the maximum fall of rain in all possible time intervals is taken into account.

Second.—Because the element of uncertainty relating to the rate of rain-water inflow into storm-water inlets is confined within narrow limits, and the possible error in the selection of a reduction factor is correspondingly small.

Third.—Because the essential element, the increase of water storage in the conduit system during a critical period of time, is subject to determination with reasonable accuracy.

Fourth.—Because it is simple and generally applicable.

Storm-Water Relief Outlets and Sewer Capacities.

The ordinary principle of providing storm-water relief outlets from the sewers of the combined system, where opportunity offered has been adhered to in designing the storm-water conduits for San Francisco. The infrequency of rain in excess of ½ in. per day made it appear desirable to let the relief outlets come into service only when the rainfall rate exceeded this amount. In other words, the sewers were planned to carry the run-off due to a rain of ½ in. in 24 hours, to the sewerage outfall point. The remainder of the flow during storms will pass through the relief outlets on the shortest course to bay or ocean.

The sewers of the combined system, therefore, were given a capacity above relief outlets to carry:

The sewage proper;

A certain amount of soil-water, entering by leakage;

The run-off due to a rainfall of greatest intensity;

Some allowance as a safety margin.

Below relief outlets:

The sewage proper;

A certain amount of soil-water, entering by leakage;

The run-off due to a rainfall at the rate of ¼ in. in 24 hours.

The sewer capacity thus determined for points below relief outlets is nearly four times as great as the mean flow of sewage proper. The storm-water reliefs, therefore, should not come into service with more than 25% of sewage in their outflow, and this amount only at times when their outflow is small. When in service at full capacity the sewage carried by them will be so dilute as to be quite inoffensive.

Sewage Disposal.

Dilution and Outfall Points.

Consideration was given to the various methods of sewage disposal which have been developed and found satisfactory.

Utilization by application to land was entirely out of the question, owing to San Francisco's location at the extreme point of a narrow peninsula, and the remoteness of lands which would be suitable for cultivation under sewage irrigation.

Treatment in filter beds, to produce an inoffensive effluent, or chemical treatment for like purpose, would entail much additional cost and is not at present necessary for any part of San Francisco's sewage, owing to the great dilution that can be secured, and the favorable set of the currents from the selected outfall points away from the shore.

Dilution in the waters of the bay and the ocean is without doubt for San Francisco the natural and proper method of sewage disposal.

The questions to be considered, therefore, relate to the best points of outfall and the best method of conveying the sewage to those points.

Without discussing at length the studies of tidal currents made in 1893 with floats, and the results of current measurements made from anchored boats by the United States Coast and Geodetic Survey some years before, the conclusions reached will be briefly stated.

The float observations did not cover the entire water front, nor all possible stages of the tide. The means were not at command to accomplish this. Generally known facts or opinions relating to the direction and velocity of the bay currents led to the selection of certain points from which the floats were started at various stages of the tide and then followed day and night by boat until stranded or well started seaward. The float was of wood about 6 ft. long, weighted at one end and carrying a flag or lantern at the other.

Float observations were made from points off Hunter's Point, Potrero Point, Center Street Pier, and Powell Street Pier. At each of these points floats were started at low tide, at high tide, and at intermediate stages. There were no float observations made oceanward from Golden Gate as these did not seem necessary. The current in and out is swift. Someone has computed that 1 cu. mile of water passes through Golden Gate each way every day. This vast volume, as it flows outward, entrains to some extent the water that lies off the beach, southward from Point Lobos. This is probably the reason why the ocean beach is exceptionally free from drift.

All the Hunter's Point flood-tide floats passed up the bay, clear of tide marsh and shore line. The ebb-tide floats cleared Potrero Point by more than half a mile and passed east of Mission Rock.

Floats started at flood tide from the Sugar Refinery wharf at Potrero Point took a course inside of a line drawn from Potrero Point to Hunter's Point; one was caught and held in the flood-tide eddy south of Potrero Point. At ebb tide all floats passed to the east of Mission Rock into the deep water of the bay.

The floats started at flood tide from the wharf of the Pacific Rolling Mill, farther north on Potrero Point, all hugged the shore toward Hunter's Point. The ebb-tide floats showed equally unfavorable conditions. Their course was close to the end of Center Street pier, to the westward of Mission Rock.

Floats started at flood tides from the end of Center Street pier, oil wharf, passed close to Potrero Point. Those at ebb tide moved toward Mission Rock.

The floats started at the end of Powell Street pier on flood tides in part took a course somewhat off shore into deep water, in part they went gradually shoreward. The ebb-tide floats all went off shore more or less, most of them clearing Black Point by more than one-quarter mile.

The conclusion reached from the foregoing observations was that sewage might safely be discharged into the bay or ocean on the northern frontage of the city, provided only that the outfall be arranged far enough off shore in deep water.

The eastern frontage of the city, from near Market Street to near Hunter's Point, is not a desirable locality for disposing of sewage. It was recognized, however, that the discharge of sewage in limited amounts through submerged outlets in deep tidal waters is rarely objectionable, and, therefore, that small shore areas which cannot at once be made tributary to the contemplated main sewers, may, with proper restrictions, be allowed to discharge into the bay.

Hunter's Point was found to offer exceptional advantages for a delivery of sewage into swift tidal currents with ample and thorough dilution.

The following points of outfall were finally selected:

Off North Point, with discharge in at least 36 ft. of water at 1 200 ft. or more from the water-front line.

Off Hunter's Point, in a corresponding depth.

Off the ocean shore just west of Baker's Beach, at the foot of 27th Avenue.

Off the foot of Scott Street, in at least 36 ft. of water, and well off shore.

Two other points were selected, though not to be utilized at once. These are Fort Point, where it may be found desirable after many years to concentrate all sewage reaching the northern city front, and Point Lobos, where it was foreseen that the sewage of the Sunset and other ocean-slope districts should ultimately be delivered.

Of the areas tributary to the selected outfall points it may be said:

The North Point main is to receive the sewage from an area of about 11 500 acres, which includes the greater portion of the residence district of the city and the entire business section. About 80% of the city's population was within this area at the time the report was written in 1899.

The territory tributary to the Hunter's Point main comprises 2 300 acres.

The Point Lobos outfall will ultimately receive the sewage from the greater portion of the ocean slope of the city.

The Scott Street outfall will serve the Harbor View district, until it becomes desirable to extend the North Point main to Fort Point.

The conclusion having been reached that the best points for the discharge of sewage are the projecting points of the water front which are swept by swift tidal currents, it became necessary to give Fort Point special consideration. Fort Point, at the south side of Golden Gate, is without doubt the best location for an outfall, but, for the present, it is inaccessible owing to remoteness from the built-up section of the city, and the great cost that would be involved in reaching it in advance of general water-front improvement.

In the order of their desirability as points of outfall for the sewage of bay-slope districts there seems to be no question that Fort Point should be ranked first, though, as explained, not now available; North Point, second, and Hunter's Point, third.

Consequently, the collecting system of sewers has been planned so as to make the greatest area possible tributary to the North Point outfall, and let Hunter's Point take care of the remainder.

This is the fundamental reason why, as elsewhere explained, the upper portion of the Islais Creek drainage is to be cut off from the Hunter's Point system and made tributary by gravity flow to the North Point main, by tunneling along San José Avenue under College Hill. By this arrangement, moreover, an immediate installation of the outfall sewers for an important outlying district becomes possible, and it has the further advantage of reducing the amount of sewage to be pumped over from the Hunter's Point main into the North Point sewer system if, in the future, it ever becomes necessary to abandon the Hunter's Point outfall. Furthermore, it appears unobjectionable to permit the temporary delivery of sewage in small amounts at other points of the bay front, provided the discharge be always effected well off shore in deep water.

Sewerage Districts.

Sewerage districts were thereupon laid out in conformity with these principles and the plans for the systems of collecting conduits were completed.

The areas noted in Table 16 and the estimated population, which is really the assumed distribution of population when San Francisco has reached the 1 000 000 mark, were used as guides in estimating, by methods already described, the quantity of sewage, soil-water, and storm-water to be cared for in the sewers.

TABLE 16.—San Francisco Sewer Districts.

Name of district and sub-district.	Area, in acres.	Estimated future population.
North Point District:		
County Line West	836	24 800
County Line East	381	8 500
Persia Avenue	246	7 300
Sunnyside	579	21 700
Glen Park	582	17 500
Silver Avenue West	177	5 300
Valley Street and Upper Army Street	794	31 700
West Bernal Heights	149	6 000
Lower Islais Creek	1 125	47 900
West Potrero, Upper Twenty-second Street	620	31 000
Eighteenth Street	660	33 000
Fourteenth Street	1 307	74 500
Miscellaneous	27	1 600
Eleventh Street	978	54 400
East Potrero	334	16 700
Fifth Street, Sixth Street, Seventh Street and Miscellaneous	782	103 700
Kearny Street	41	6 200
Mission Flats	845	72 500
North Rincon Hill	126	15 700
Portsmouth	157	31 000
Yerba Buena	279	33 500
North Beach	500	30 000
Totals	11 525	674 500
Hunter's Point District:		
College Homestead	122	3 700
Gaven Street	68	2 000
St. Mary's	91	3 600
South Bernal Heights	123	4 900
University Mound	488	14 600
West Silver Terrace and Miscellaneous	65	2 000
North Silver Heights	77	3 100
Railroad Avenue	115	4 600
North Slope, Hunter's Point	306	12 200
Bay View	590	23 600
South Slope, Hunter's Point	249	10 000
Totals	2 294	86 300
Harbor View District	816	48 100
Richmond	1 200	60 000
West Richmond	212	19 200
Upper Sunset	1 658	64 200
Lower Sunset	2 564	98 600
Ocean View	783	29 800

Population, Water Consumption and Sewage.

Before describing the district systems, a word is to be said relating to the prospective growth of San Francisco, and the water consumption. The population estimate, as made in 1899, is shown by the diagram, Fig. 11. For comparison, the population of several other cities of the United States is also shown. The conclusion was reached that about 50 years would elapse before San Francisco would have a population of 1 000 000. Fifty years being a reasonable time for which to insure adequate service to all parts of the city; and fully realizing that no

forecast now made for a period so remote is to be regarded other than as a probability, this population was accepted as that on which estimates of water consumption and, therefore, volume of sewage proper and the like, should be heard.

The information presented in Fig. 11, and extended to include the results of the census of 1900, is shown in Table 17.

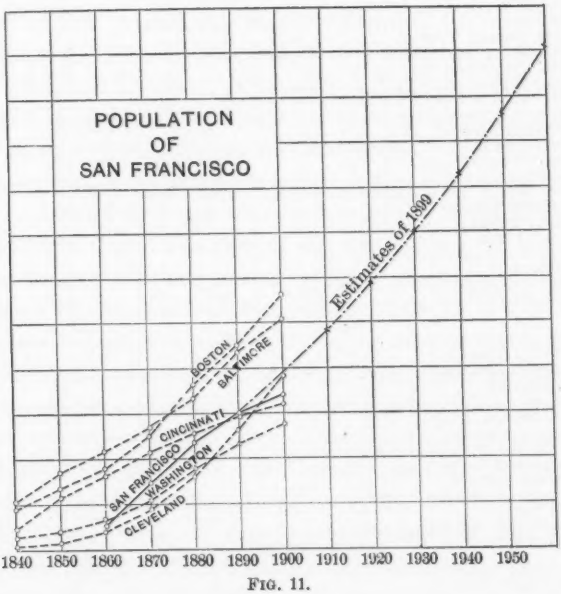


FIG. 11.

TABLE 17.—POPULATION OF CITIES FROM UNITED STATES CENSUS, AND ESTIMATED.

Year.	Washington.	Cleveland.	Cincinnati.	San Francisco.	Baltimore.	Boston.
1840	23 364	6 071	46 338	.....	102 313	93 383
1850	40 001	17 034	115 435	.....	169 054	136 881
1860	61 122	43 417	161 044	56 802	212 418	177 840
1870	109 199	92 829	216 239	149 473	267 354	250 526
1880	177 624	160 146	255 139	233 950	332 313	362 839
1890	230 392	261 383	296 908	298 997	434 439	448 477
1900	278 718	381 768	325 902	342 782	508 957	560 892
1910	.....	.....	.....	*480 000	.....	.....
1920	.....	.....	.....	*590 000	.....	.....
1930	.....	.....	.....	*700 000	.....	.....
1940	.....	.....	.....	*830 000	.....	.....
1950	.....	.....	.....	*960 000	.....	.....
1960	.....	.....	.....	*1 100 000	.....	.....

* Estimates made in 1899.

As a result of the destruction of the business section and a large part of the residence district of San Francisco by fire in 1906, there was at that time a sudden drop in the actual population curve.

If recast at the present time there would, no doubt, be some departure from these estimates.

From information available in 1899, it was found that, exclusive of water pumped from bay or ocean for the supply of bathing tanks, for use in street sprinkling, and for use in condensers, about 70 gal. per capita per day were supplied to the city by the Spring Valley Water Company. The maximum daily per capita consumption at that time was 82 gal.

It is believed that San Francisco's cool climate, particularly in the summer, the compact development of the residence districts, with restricted lawn and garden areas, and the natural difficulties in the way of bringing in a copious supply, will always compel more than ordinary economy in the use of water. It is not likely, therefore, that the present consumption of 75 gal. per day will ever be greatly exceeded. Allowing 20% as the excess of the maximum daily consumption over the mean, and making a small allowance for water taken from ocean, bay and private wells (4% was assumed), the conclusion was reached that 94 gal. per day should be taken into account in estimating the volume of sewage proper.

About 75% of the water used for all purposes in the city will reach the sewers. But the rate at which the water is used and delivered to the sewers is not uniform. It was assumed, without special local studies to confirm the accuracy of this assumption, that the maximum flow would exceed the mean by 50%.

Based on these premises, it was estimated that for each 1 000 inhabitants the average quantity of sewage proper, on days of maximum water consumption, would be 0.109 sec-ft., and that the maximum rate at which this would reach and be carried off by the sewers would be 0.163 sec-ft. This was the unit adopted for use in estimating the required capacity.

Soil-Water.

The quantity of soil-water entering the sewers by leakage depends on the care with which the sewers are constructed, their permeability, leaky joints, and also the relative elevation of the water-table and sewer. When the water-table is below the sewer there may be leakage from the sewer into the ground, but there will be no reverse flow.

Throughout a large portion of San Francisco the sewers will lie above the soil-water plane. If winter rains bring the water-table up to cover the sewer, in the high parts of the city, it will be for short periods only. Therefore, very little inflow from the soil into the sewers is to be assumed, except in the low marginal and down-town areas, where the plane of saturation lies above the sewers. It would be interesting to know whether this plane could not be permanently lowered, in view of the fact that the bay front of the city is, or was, skirted by mud flats which are to be considered nearly impervious. But this cannot be ascertained short of very expensive experiments.

If there is a great leakage into the sewers, they may have some effect on the water-table, and the leakage will gradually decrease until equal to the ground-water replenishment, from whatever source this may come. In the absence of adequate information relating to ground-water depth in different parts of the city, a uniform allowance for an increment of the flow in the sewers due to this cause was made for all the high portions of the city and a larger allowance for the low areas. The ground-water reaching the sewers from high ground was introduced into the calculation of sewer capacity at 0.001 sec-ft. per acre, and that from the low parts of the city, where the sewers, for the most part, will lie below the natural soil-water plane, at 0.003 sec-ft. per acre.

The Tides of San Francisco Bay.

The range of tides in San Francisco Bay is thus referred to in the *Pacific Coast Pilot*, by George Davidson, Hon. M. Am. Soc. C. E., who for many years was in charge of the work of the United States Coast and Geodetic Survey on the Pacific Coast:

"From the lower low water ('low water large') the tide rises for about 7½ hours, say 4.4 ft. to the smaller of the two high tides ('high water small'), then falls 1.4 ft. in less than 4½ hours to the 'low water small,' which is higher than the preceding low water; then rises say 2.9 ft. in 6¼ hours to the higher high water or 'high water large;' it then falls again 5.8 ft. in over 7 hours to the lower low water, or 'low water large.'

"Instead of the above figures, the fall from high water small, or 'half tide' to the 'low water small,' may range from 3½ ft. at one position of the moon to 0.3 ft. at another; in the latter case there will be apparently a long stand of about 5 hours.

* * * * * * * * *

"The average difference of the higher high and lower low waters of the same day is 5.2 ft., with a greatest observed range, noted for February 8th, 1876, of 9.93 ft."

The "Tide Tables" for 1899 of the United States Coast and Geodetic Survey gave the following information:

The expected mean range of tides is 4.5 ft.
The expected mean range of spring tides is 5.4 ft.
The expected mean range of neap tides is 3.5 ft.
The expected mean range of great tropic tides is 7.2 ft.
The expected maximum range of tides in 24 hr. is 7.8 ft.
The expected minimum range of tides in 24 hr. is 3.1 ft.

From these tidal ranges, and the area of the bay, together with the area and elevation of the adjoining submersible land, and with proper allowance for the difference in the time at which the high and low tides occur, the total volume of flow through the Golden Gate can be estimated.

To the outflow through the Golden Gate, however, there is to be added the flow of the rivers tributary to the bay. This is not inconsiderable.

It is the run-off from more than 60 000 sq. miles of tributary area. The two rivers, the Sacramento and the San Joaquin, contribute more than 4 000 sec-ft. at their lowest autumn stage, and more than 80 000 sec-ft. throughout the first six months of the year, often reaching from 150 000 to 200 000 sec-ft. for short periods of time.

River flow through the bay alone would effect sewage dilution from 30 to 1 500 fold, yet the river water is only a trifle when compared with the great return flow of waters which have entered the Golden Gate on a rising tide.

North Point Sewer District.

The North Point Sewer District has an area of 11 525 acres. Its estimated population, when that of the city, of present area, is 1 000 000, will be 674 500.

About 80% of the city's population in 1899 lived within this district. It includes the northern frontage of the city easterly from Black Point, the eastern frontage as far south as the hills on the south side of Islais Creek, and extends inland to the summit of the peninsula ridge and along the same, south to, and slightly beyond, the county line, including all the Islais Creek water-shed already described.

Nearly all this district is to be sewered on the combined system. The old sewers are utilized to the greatest extent possible.

The position of the main intersecting sewer was determined by two governing points, the outfall at North Point and the point on the old line of Mission Creek where the sewage of the Mission is to be intercepted. The sewer was placed low enough to serve the district near Harrison and Fourteenth Streets. At that point many street grades have been established without due regard to future requirements, and to the permanent injury of adjoining property. Improvements of street surface and improvements on private land have been made to conform to these grades, many of which are too low. It is doubtful whether material correction of such grades will ever be possible. The North Point main, therefore, was planned to fit the grades as established, the aim having been to keep the hydraulic grade line at least 1 or 2 ft. below the street surface at the critical points. This was the best that could be done for this district which, under present conditions, is submerged whenever there is a heavy downpour of rain. In order that the reason for small safety margin, for the scant elevation at a few points of street surface above the hydraulic gradient of the full conduit, may be better understood, it may be stated that there were such points, as at Eighteenth and Division Streets, where the grades marked out a pot-hole arrangement. The official grade there at the time the sewer system was planned was 7.5 ft. above city base,* while at all street intersections around this point the official grades ranged from 10.5 to 13 ft. At Thirteenth and Harrison Streets there is a similar arrangement. Street grades of 6 and 6.5 ft. are entirely surrounded by official grades several feet higher. Official street elevation is similarly defective at the northern termination of York, Florida and Alabama Streets, and at the intersection of Fifteenth and Shotwell Streets; also at Sixteenth and Shotwell Streets; at the eastern termination of Fifteenth Street, on Folsom Street midway between Sixteenth and Seventeenth Streets, and at Fourteenth Street; and there are still other points where grades should be changed so as to eliminate completely the pot-hole feature.

In the past this district has suffered one inundation after another. The proposed increased capacities of the sewers and storm-water drains

* The grade at this point was raised a few years after the report was written.

will greatly ameliorate this condition, but the mischief done by the establishment of faulty street grades cannot be entirely undone by the construction of sewers, and it must be left to the property owners to find out how much better off they would be if the high-water line in the sewers could be kept 5 or 6 ft. below the street surface instead of rising almost to it. The only satisfactory relief from the local floods that may result from defective or choked storm-water inlets and from the annoyance of water in cellars, or, at least, of interrupted cellar drainage during severe rain storms, will result from raising the street grades, particularly at the points where pot-hole features are to be eliminated. The present property owners are often the innocent sufferers, from the desire of their predecessors to have official grades established at natural ground height or but little above it, in order to save the cost of grading.

The head of the North Point main, if considered as an interceptor, is far out on the Mission Road, at the point, in fact, where this road crosses Islais Creek. From this point the sewer will be in tunnel for a distance of 4 500 ft. following the line of Mission Road and San José Avenue.

At Army and Valencia Streets the sewage now flowing down Army Street sewer is to be intercepted and diverted northward. The present sewer eastward from this point will continue in service as a storm-water relief outlet, and as a main from which a mile farther east sewage will be pumped to the nearest point of the North Point main.

The course of the North Point main is almost due north following the streets best adapted by reason of elevation to receive it. It will be on Twenty-sixth Street from Valencia Street to Treat Avenue, on Alabama Street from Twenty-fifth to Nineteenth Streets, and will reach Division Street at Eighteenth Street. It will follow Division Street to Harrison Street, paralleling the present Channel Street sewer; thence it will be on Harrison Street to Eighth, on Eighth to Howard, thence on Howard and New Montgomery or Second Street northwest to Market Street and north to the water front on either Montgomery or Sansome Street. All along its course it will intercept the sewers entering it from the west.

At Eighteenth and Division Streets, at Fifteenth Street, and about on the line of Fourteenth Street, will be storm-water relief outlets of large capacity. The outlet arrangement at Eighteenth Street is practi-

cally that of two parallel conduits with a common roof, separated from each other by a dividing overfall weir. The entire flow will be confined by this weir to the North Point main until the full capacity is reached, whereupon the excess flow will go over the weir into the storm-water conduit.

Other storm-water outlets will be provided at Seventeenth, Sixteenth, Eleventh, Seventh, Sixth, and Fifth Streets, also at Mission Street and at Commercial and Jackson Streets, as well as a final one at the North Point screen-house, where the sewage will enter the outfall pipes.

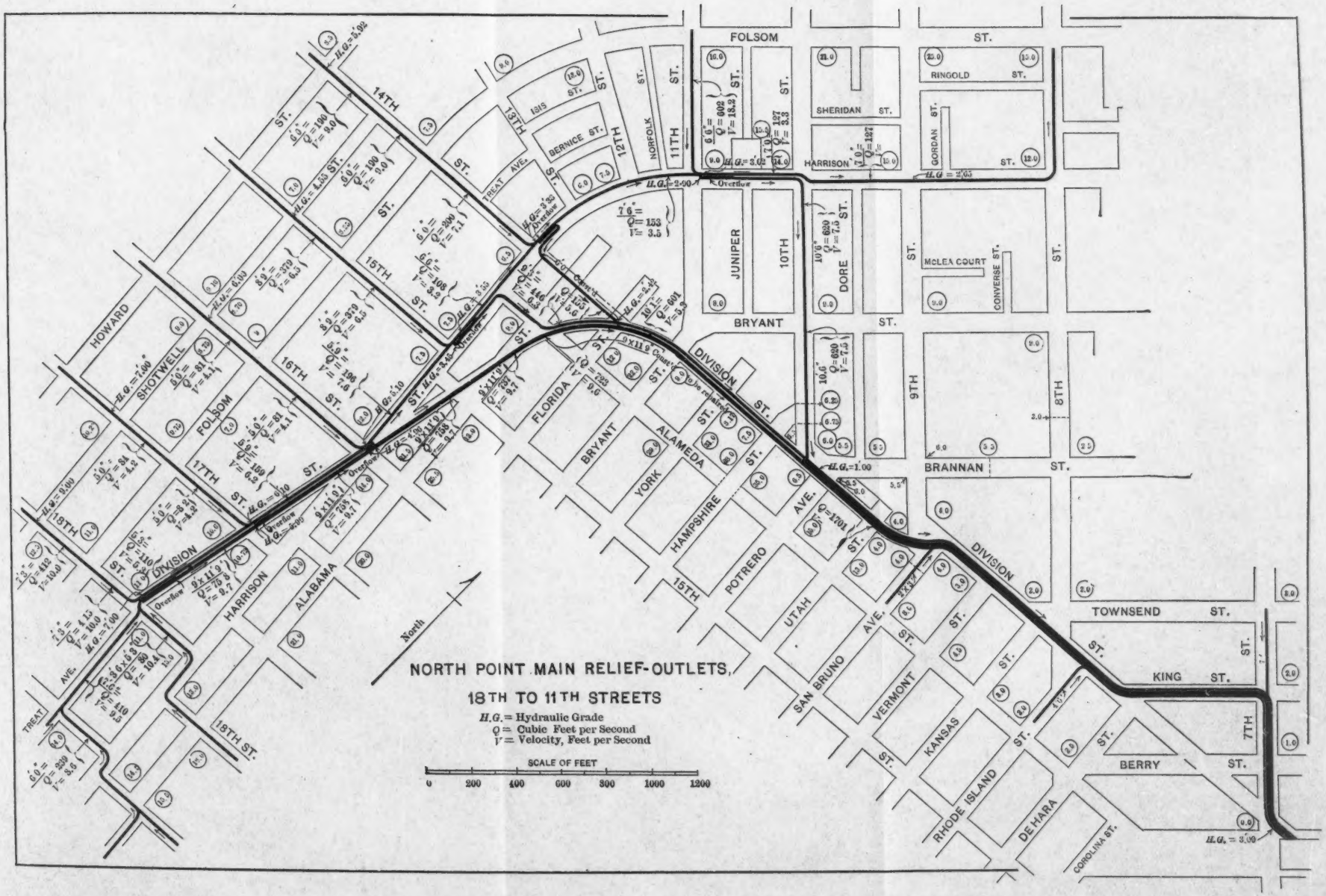
Large conduits will carry the water from each of the storm-water outlets in the most direct line to the open water in Channel Street, or to the bay. Of these conduits, the only one of special interest is the Division Street conduit which will have its head at Eighteenth and Division Streets, but will be the recipient also of the storm-water overflow at Seventeenth, Sixteenth, Fifteenth, and Fourteenth Streets, and at several other points along its course to the head of the open water in Channel Street.

This conduit, shown on Plate IX, will be called upon to carry ultimately about 760 sec-ft. near Eighteenth Street, and about 1 700 sec-ft. at its outfall end. It will be a conduit 9 ft. high and 11 ft. 9 in. wide from Eighteenth Street to Tenth Street, being paralleled by a second conduit from Alameda Street to Tenth Street. Thence to its outfall, for a distance of about 2 000 ft., the conduit will be constructed with two or three compartments. Three, each 9 ft. wide, will be required ultimately. It may be found advisable, however, to construct only two of these at the outset. Owing to the fact that the floor of the conduit must be kept as high as possible on account of the unfavorable character of the ground along its route, and because the street surface along the line of this conduit is lower than desirable, scant headroom is available, and the conduit has been planned with a flat top of reinforced concrete.

It has been suggested by interested citizens that the main storm-water outfall should be at the foot of Sixteenth Street, and by some it is even contended that the sewage of the entire area that can be made tributary to the intersection of Sixteenth and Division Streets should be carried in a large conduit along Sixteenth Street to the bay.

Taking Fourteenth and Harrison Streets as the initial point for

PLATE IX.
PAPERS, AM. SOC. C. E.
MARCH, 1909.
GRUNSKY ON
THE SEWER SYSTEM OF SAN FRANCISCO.



such an outfall, it can readily be seen by a glance at any map of the city that the distance thence to Sixteenth Street and along Sixteenth Street to the end of Center Street wharf would be twice as great as the distance from the same point along Division Street to the head of the open water in Channel Street. Consequently, a conduit along Sixteenth Street would have less gradient than a storm-water outlet along Division Street. It would have to be of larger cross-section, and the velocities in it would be less than if located along Division Street. In other words, for conduits of similar dimensions, on the two routes, there would be more probability of surface inundation at and near Fourteenth and Harrison Streets if the conduit were located along Sixteenth Street than if located as planned.

The foot of Sixteenth Street does not meet the requirements of a permanent outfall for the sewage of the area that can be made tributary to it. Nothing that can be delivered at North Point should be allowed to reach the water at such a central point on the eastern bay frontage.

Attention may be called to the fact that, in line with this requirement, the sewage of the eastern portion of the Potrero Hills is to be collected in a secondary interceptor which will skirt the northern base of these hills and connect with the North Point main at Ninth and Howard Streets. The storm-water overflow from this secondary East Potrero interceptor will flow in suitable conduits to the Division Street relief outlet.

The North Point main will be in tunnel under Telegraph Hill for 2 500 ft., if it follows Montgomery Street. If located on Sansome Street, the tunnel construction, if required at all, would be much shorter.

In its largest section, the sewer was planned to be circular, 8 ft. in diameter, with a concrete invert and a brick arch. In doubtful ground it is to be supported on piles. Double sheet-piling on each side is to facilitate trenching. The piles used as guide piles for the sheeting are to be left in place as a part of the sewer foundation, and the sheet-piling on either side of the trench is to serve as a part of the form for the concrete invert. Longitudinal steel rods are to be placed in the concrete, and as many piles as may seem necessary for support are to be driven in the trench and capped with the concrete of the sewer invert.

The hydraulic gradient prescribed for the sewer when flowing

full will have an elevation of 7.00 ft. above City Base at Eighteenth and Division Streets, dropping to 3.50 ft. below City Base at the outfall. Its gradient is about 0.0005, or 1 in 2 000. The sewer invert, as originally planned, was to have gradients gradually changing from 1 in 1 800 near Division and Eighteenth Streets to 1 in 2 000 at the outfall.

The hydraulic grade line at the North Point screen-house will be 0.50 ft. above extreme high water of the bay and 3.20 ft. above ordinary high water. The sewer invert at the same point will be about 12.90 ft. below City Base, or a little more than 6 ft. below ordinary high tide.

During the rising tide, therefore, the discharge of sewage will be retarded; during the falling tide it will be accelerated. The effect of this will be a reduced outflow into the flood tides and an increased discharge into the ebb, which is a desirable feature.

The area of the city on the bay front which lies to the eastward of the North Point main will be treated in three subdivisions, from each of which sewage will be pumped into the North Point main.

It is not proposed in this paper to enter upon a minute description of the system of sewers outlined for these and for the many other main and sub-districts, because the problems are in large part the same as are presented in all growing cities. Only those features of the problem will be referred to which appear somewhat unusual or may otherwise be considered of special interest.

Yerba Buena and Mission Flats Sub-District.—The Yerba Buena sub-district, which extends from Telegraph Hill on the north to Rincon Hill on the south, and lies between the North Point main and the bay, is a low, flat, for the most part filled-in, section of the city. It is throughout an important business district.

The storm-waters of this sub-district are to be delivered to the bay for the most part in the sewers now in use, but remodeled as far as practicable to give them continuous slope toward their outfall points.

Sewage proper will be provided for in separate conduits leading to a pumping station between Commercial and Sacramento Streets, a short distance east of Drumm Street. The pumps will send the sewage westward into the North Point main.

The Mission Flats sub-district, 845 acres in extent, which embraces the old Mission Bay region as far as it lies below or east of the

North Point main, will also be sewered on the separate system. The pumping station for this district is to be located north of Channel Street, near Fifth Street. The sewage south of the Channel will be carried under the open waterway in an inverted siphon. The pumps will send the sewage northwestward along Fifth Street into the North Point main.

In both these districts the sewers of the separate system will be ironstone pipes. The smallest lateral sewers are to be 8 in. in diameter and the minimum gradients on which these are to be laid, with but few exceptions, will be 1 in 200.

The principal objection to the use of the combined system in these districts is the fact that the surface of the districts lies low. Very much of it is at City Base. The efficient drainage of basements has not been possible with the combined system heretofore in use. Better service was demanded. It was necessary to choose between, on the one hand, a combined system of sewers leading to pumping stations, with storm-water relief outlets to the bay and intermittent pumping, and, on the other hand, the complete separation of the sewers from the storm-water conduits. In the former case there would have been times, during rain storms, when the sewers would have been full to a plane well above high tide in the bay, and back-flow into basements unguarded by check valves would have been inevitable. From this situation there was no remedy except either to provide pumping capacity on a combined system great enough to handle the maximum storm-water flow, or to introduce the separate system. The latter plan was adopted, but it, too, has its disadvantages. One is the duplication of conduits in streets already crowded with pipes for gas and water, and with conduits for telephone and telegraph wires and wires for transmitting electricity. It was found necessary in many places to provide sewers for each side of the street.

The main drawback to this system is probably to be found in the fact that some of the streets of the filled-in areas are still in a condition of subsidence. On this point there was very little information available in 1899. It was known that the lower end of Market Street, which had been paved many years before at official grade, was then several feet low, and the various buildings with their sidewalks, which had been constructed at grade, had settled measurably. Nothing was known concerning the continuance of the subsidence. The general

opinion seemed to prevail that but little more was to be expected. The adopted system, under these circumstances, included provision for the support of the sewers on piles. It was proposed to place the sewers in a continuous, pile-supported bed of reinforced concrete wherever the ground was doubtful.

In the light of later information, it now seems uncertain whether any type of construction can be devised that will hold the sewers at grade.

As City Engineer during 1900 to 1904 the writer made studies of street subsidence. Careful levels were taken at intervals of one year. The results of these studies for the two years, 1901 to 1903, are shown in brief form in Table 18.

TABLE 18.—SUBSIDENCE OF STREETS IN SAN FRANCISCO.

Location.	SUBSIDENCE, IN FEET.	
	1901–1902.	1902–1903.
Davis Street, Market to East Streets	0.05	....
Jackson Street, Montgomery to East Streets	0.03	....
Spear Street, Market to Bryan Streets	0.05	....
Mission Street, First to East Streets	0.05	0.02
Harrison Street, Fourth to Seventh Streets	0.15	0.19
Sixth Street, Howard to Brannan Streets	0.05	0.05
Sixth Street, Brannan to Channel Streets		0.08

From April, 1902, to April, 1903, there was no appreciable subsidence on Davis Street except a small amount at its intersection with Vallejo Street, where the subsidence during the preceding 12 months had been 0.08 ft.

From April, 1902, to April, 1903, there was no appreciable subsidence on Jackson Street except a small amount at Drumm and East Streets, where the subsidence during the preceding year had been 0.06 ft.

From April, 1902, to April, 1903, there was subsidence on Spear Street only at Mission Street, where the subsidence during the preceding 12 months had been 0.06 ft.

The subsidence is not confined to the streets alone, but includes adjacent buildings; even those within the area under consideration that are supported on piles 90 ft. and more in length show measurable subsidence.

The lower end of Market Street in 1903 was from 2 to 3 ft. below

official grade from curb to curb, the street-car tracks supported on piles having settled with the remainder of the street.

As subsidence of the character here referred to includes every structure in or adjacent to the street, no provision for the support of sewers would be adequate to prevent settling. If the sewers are laid to grade there will be sections dipping below grade after a few years, and the dip will increase until after a time the departure from the true gradient may be so great that reconstruction of some of the down-town sections will be necessary.

The earthquake of 1906 has occurred since the subsidence studies were commenced. No information is at hand at this writing as to the effect which the violent shaking has had upon the general elevation of the street surfaces in the made-land districts. Local upheavals of a foot or more, and local depressions of the ground's surface have been noted in various parts of the city. Where these occurred they were more or less destructive to all conduits in the streets, and would have caused serious derangement of the sewer system as planned if it had been in service. It is probable that whatever changes have been produced in the street levels, by the earthquake, have brought them nearer to a permanent condition of stability. At any rate, the probability of the recurrence of earthquakes of sufficient force to prove destructive to sewers is too remote to be taken seriously into account in planning the system. The location of the areas where a derangement is most likely to occur, however, has been pretty clearly indicated by the breaks in conduits of all kinds in the earthquake of 1906, and this knowledge may here and there lead to some modification of the location of a sewer or to some modification of design.

Lower Islais Creek District.—A dual treatment has been planned for the Lower Islais Creek district. From this district sewage proper is to be pumped through a long force main into the North Point sewer. Where there are no sewers now in use the collecting system will be separate from the storm-water drainage; but where sewers have already been constructed to serve for sewage and storm-water combined these are to remain in service. The arrangement at the pumping station, which will be centrally located, will be such that excess storm-water will pass through a relief outlet to Islais Creek. Whenever the excess flow is long continued, which will be coincident with a full North Point sewer, the pumping from the sewers on the combined

system will be stopped. The sewers of the separate system are to reach this pumping station at 15 ft. below City Base. This low grade is made necessary by the large tributary area with low-surface elevation. At a mean stage of water in the receiver at the pump the total lift, including friction in the discharge main, will be about 50 ft.

Only those sewers of this, as of other, districts that lie within improved territory or that are at once required to reach pumping stations or outfall points were recommended for immediate construction. There are some cases in which the sewers recommended for immediate construction lie along streets crossing the marsh and not now graded. In these cases it was assumed that such streets would be ordered graded as a matter of urgency, and that within a few years they would be ready to receive the sewers.

The North Point Outfall.—The screen-house at the outfall of the North Point main is to be a neat structure of brick or concrete. There will be two settling basins of small diameter, to intercept coarse sand and the broken rock reaching the sewers occasionally with the storm-water from the macadamized streets of the outlying regions. These basins are to be arranged so that one or both may be in service. The removal of material from them will be by dredging, possibly by the hydraulic method.

Alongside of the main sewer, just before it enters the screen-house, there is to be a long overfall weir, with crest at the hydraulic grade line, or a little below this, over which any excess flow of storm-water will drop into a low, flat conduit of brick or concrete leading directly to the bay.

The screen-house will be equipped with screens for the interception of rags, cork, and other floating material that should be removed from the sewage before it is discharged into the bay waters. The outfall pipes are to be connected with the settling chambers, and are to be carried at least 400 ft. beyond the head of a solid pier projected for this point. They were planned to be of cast iron, 5 ft. in diameter. It may be found desirable to substitute reinforced concrete. They are to be placed in a trench excavated by dredging to about 50 ft. below low tide.

Hunter's Point Temporary Outfall.

In the case of Hunter's Point main sewer it is not practicable, owing to the unimproved condition of the streets and their location,

across marsh surface and points of hills, to carry the sewer at once to its final point of outfall. The improvement of the streets along which the sewer is to be built is too far in the future. Their grading for sewers is out of the question on account of cost. The main, therefore, will terminate for a time on the north shore of Hunter's Point, and a discharge will be effected through a submerged pipe well off shore.

Harbor View Sewer District.

The time may come when the demand for keeping the waters of the bay absolutely free from sewage will be so great that, whether offensive to the senses or not, the delivery of sewage on the eastern and northern bay frontage of the city must be stopped. Should this time ever come, the sewers as now planned will remain in service, but pumping will be necessary to send all sewage along the water front to an outfall at Fort Point. The arrangement can there be made to discharge practically all sewage of the bay slope of the city into the ebb tide. In this event the sewage from so remote a region even as Bay View would be delivered by pumping into the North Point main, and would be rehandled by the main pumps located at or near the site of the proposed screen-house. Until this time comes, there will probably always be more or less sewage sent into the bay at points less desirable than those selected as main outfall points.

Until actual experience shows it to be undesirable to send limited amounts of sewage into the bay with delivery into deep water, at a few selected points, the great expense of concentrating all at Fort Point need not be incurred.

It was planned, therefore, to deliver the sewage of the Harbor View District, lying on the northern front of the city, to the westward of Black Point into the bay at the foot of Scott Street. The sewers of this district, as at present, will carry both sewage proper and storm-water. Storm-water relief outlets are planned with a view to bringing existing sewers into use at their full capacity. The drainage system of the district was designed with special reference to the present improvements. The intercepting sewers follow streets along which immediate construction is practicable. When the reclamation of the waterfront property has here been completed, a second line of intercepting sewers of small capacity will be needed. The storm-water relief outlets will serve as local sewers below the upper intercepting line until they

reach the waterfront, where their ordinary flow will follow the lower interceptor and be carried by the same to the main outfall point.

This system of sewerage will have the disadvantage, for the low area close by the waterfront (as yet but partially improved), of affording only imperfect basement and cellar drainage. But the property owners will know the limitations, and can plan their improvements to meet the conditions.

The discharge into the bay will be through a large pipe, about 800 ft. long, terminating in water 40 ft. deep. The capacity of this pipe will be adequate for the ordinary sewage flow, with the usual included small amount of rain-water. All excess flow, as already stated, will go to the bay through storm-water relief outlets.

Richmond Sewer District.

For the Richmond District a sewer project was adopted some years previous to the studies of 1899. The sewage and rain-water are carried in common conduits to an outfall just west of Baker's Beach. At the time this outfall was inspected no trace of sewage at the shore nor of offensive odor could be detected. The strong tidal currents sweeping this part of the shore warrant its use as a point of disposal. The later designs provide for an extension of the outfall farther off shore into deeper water and a completion of such mains as have not yet been constructed.

Other Ocean-Slope Sewer Districts.

The topography of the ocean slope of the city westerly and southerly from Richmond District made a subdivision into four additional districts appear desirable. These are the West Richmond, Upper Sunset, Lower Sunset, and Ocean View Districts. The last named includes the Lake Merced region, and extends southward to the County line.

The problem of handling the sewage of these districts, which for the most part will in time be densely populated, is complicated by the fact that the ocean beach, extending for several miles southward from the Cliff House near Point Lobos, is a pleasure ground of the people, and must be preserved undefiled.

This consideration led the writer and his associate engineers to recommend the separate system of sewerage for all these districts,

and a delivery of all sewage proper into the ocean, off Point Lobos, where it will be so diluted as to disappear completely. From this point, moreover, the oceanward flow is away from and not toward the beach.

Storm-waters were to be collected in separate conduits with outfalls into the ocean at selected points. The principal outfall points tentatively suggested for storm-waters were at the foot of J Street and at the foot of X Street.

It is understood that the plan of treatment here outlined for the Ocean Beach drainages is not to be adhered to. It is now proposed to construct at once an interceptor for both sewage and storm-water which will discharge into the ocean at the Point Lobos outfall point already referred to. Sewage will be liberated in deep water well off shore in the vicinity of Mile Rock.

The rapid improvement that is taking place on the ocean slope has made it appear undesirable to install the works for the temporary delivery of sewage from a West Richmond pumping station into the Richmond mains.

It will be possible to place the proposed main sewer of the ocean-slope districts along V Street to Forty-fifth Avenue, to T Street, to Forty-sixth Avenue, to R Street, to Forty-seventh Avenue, to J Street, to Forty-eighth Avenue, and along Forty-eighth Avenue into the tunnel under Fort Miley Heights.

A low-level sewer will follow the Great Highway northward from X Street to a junction with the main interceptor. Velocities in the low-level sewer are estimated at 2.8 ft. per sec.

All the ocean-slope districts, except portions of Richmond, were but sparsely settled in 1899, but a rate of growth was anticipated that made it imperative to provide means at an early date for a disposal of the sewage from the improved areas. At that time there were improved areas of small extent at and near the beach, both to the north and to the south of the Park, and Upper Sunset was fast becoming popular as a residence district. At Ocean View, too, in the southwestern part of the city, a portion of the built-up area was over the divide on the ocean slope and needed attention.

The ultimate point of outfall taken into consideration for this ocean-slope section of the city was Point Lobos, near Mile Rock. The small service required for a number of years, and the high initial

cost of tunneling under Point Lobos Heights, made it undesirable to attempt to reach this point with an outfall sewer at the outset. For this reason the following treatment of the ocean-slope region was recommended. This has since been modified, as already explained.

Upper Sunset District.—The region lying south of Golden Gate Park, between the Park and the Twin Peaks and Blue Mountain group of hills, as far west as Twenty-ninth Avenue, has been designated Upper Sunset District to distinguish it from that part of Sunset which lies at lower elevation and nearer the ocean.

The sewage of this district, under the plan of 1899, was to be carried northward across Golden Gate Park and delivered into the mains of the Richmond District, with outfall at the foot of Twenty-seventh Avenue, just west of Baker's Beach. The storm-waters were to be carried westward in separate conduits.

At the present time both sewage and storm-water are delivered into the Park, as a temporary arrangement, and are being used for irrigation.

Lower Sunset District.—The main western slope of the city, from Golden Gate Park southward, to near Lake Merced, is known as Lower Sunset District. In 1899 the greater portion of this region was covered by high, irregular sand dunes. Streets had long ago been laid out, but official grades remained to be established. At that time, therefore, it was premature to commit the city to a definite scheme of sewer alignment. Since then, however, this region has been one of rapid development, and no mistake was made in 1899 in recommending the immediate construction of works for sewage disposal for which there is growing need. In this district, as for Upper Sunset District, a complete separation of sewage from storm-water was recommended. This plan has now been modified, as already stated.

West Richmond District.—The region known as West Richmond District is practically a part of the Lower Sunset District, separated from the latter by Golden Gate Park. The sewage of both this and Lower Sunset District was to have been led into the receivers of a pumping station located near the ocean on the north line of the Park. It was to be pumped from there temporarily into the mains of the Richmond District, ultimately through a tunnel under Point Lobos Heights into the ocean near Mile Rock. Under the modified plan, the pumping station is rendered unnecessary.

Ocean View District.—Ocean View District is the area sloping westward from the peninsular ridge, in the southwestern part of the city, toward Lake Merced. It embraces parts of Ocean View, Lake View and Ingleside. It is a sparsely populated district, but needs outfall sewers at once because its drainage is toward Lake Merced, which is still in use as a source of water for the city and will probably be used indefinitely, as an emergency source.

The storm-waters of this district will necessarily follow natural drainage lines westward to points where they can be intercepted for diversion past Lake Merced. Some temporary works for such diversion, with permanent tunnel outfall, have already been constructed by the Spring Valley Water Company.

Sewage proper will be collected in separate conduits, and will be carried in a closed main to a discharge into the main sewers of the Lower Sunset District.

Pumping Stations.

The details for the pumping stations were worked out by Mr. H. C. Behr, Mechanical Engineer. Centrifugal pumps were prescribed for all pumping stations, although, for use at the West Richmond station, vertical, crank-driven, triple-plunger pumps were taken into account, to be used until the low delivery through the tunnel at Point Lobos could be effected. Under the modified plans there will be no West Richmond pumping station.

The receiver capacities were fixed, in a measure, to fit the available ground area. If necessary, they may be added to in the future. The receivers were planned circular in outline, in triplicate for each station, and it was assumed that all would have to be carried below the water-table, and that the pneumatic method of construction would be necessary. The receivers at each pumping station are to be interconnected and arranged for use separately or in conjunction. The sewage, before entering the receivers, is to be screened. Ventilation is to be provided by a shaft or chimney, in which a small fan blower, with a multiple injector nozzle, is to create a draft.

All pumps are to be belt-driven. Small units are suggested for the pumps in order to secure the greatest possible adaptability to all requirements. The motors were only tentatively selected for cost-estimate purposes, it being apparent that gas, oil, and electric motors are

all well suited for the work required. Steam engines require more space and a greater cost of attendance. They are objectionable on account of the smoke nuisance, and are not as well suited to the intermittent character of the work that may be required.

The force or discharge main from each pumping station is to be of cast iron. Additional facts relating to the pumping stations are given in Table 19.

TABLE 19.—Sewage Pumping Stations.

Name of station.	Required capacity, in second feet.	Total lift, including friction, in feet.	Ultimate horse-power required.	Present horse-power.	Capacity of receiver, in gallons.	Length of force main, in feet.	Diameter of force main, in inches.
Yerba Buena.....	6	31	42	35	60 000	2 100	16
Mission Flats.....	15	35	120	80	60 000	4 000	18
Lower Islais Creek.	12	65	180	88	60 000	4 700	18
Bay View.....	5	30	35	14	40 000	1 650	8

Types of Sewers.

Concerning the types of sewers to be used, but little need be said. Ironstone pipes are to come into use up to diameters of 2 ft. Sewers of the next larger size, and up to 3 ft. 6 in. by 5 ft. 3 in., are to be egg-shaped. Larger sewers are to be circular, except as stated in the case of the main relief outlet to Channel Street, which is to be flat—the lower end in two or possibly three compartments, as already explained, each nearly rectangular, 9 ft. wide and 8½ ft. high.

The egg-shaped sewers and the large circular sewers were planned to have concrete inverts with reinforcement to give added strength and stability. They were designed without vitrified brick invert lining.

It is understood that the plans have recently been modified somewhat in the matter of sewer types. The use of reinforced concrete is to be extended. All sewers of greater diameter than 2 ft. are to be constructed of this material. Vitrified brick is to be used for invert lining of some of the sewers.

It is to be stated in this connection that recent examinations by the City Engineer of concrete sewers in Sixth Street and H Street that were constructed about five years ago show that these sewers, except for a slight wear in the invert of the H Street sewer, are in excellent condition. No deleterious effect of sewage upon the concrete could be noted.

Whether the tendency, so generally apparent throughout the country, to build the sewers with concrete shells of minimum thickness, which is sure to be attended with many failures, is to be followed or not, remains to be seen.

The Bond Issue.

The report of 1899 was accompanied by a cost estimate covering the construction of the sewers and storm-water conduits immediately required. The expenditure of $4 600 000 was recommended.

At an election, held soon after the adoption of the report, bonds in this amount were voted to carry forward the work. Before these bonds were issued there was a modification of the form of government in San Francisco. Theretofore the city was subject to control by the State Legislature. Legislative enactment, in the early history of San Francisco, had consolidated City and County. Amendments to the Consolidation Act were thereafter the convenient means of securing privileges in the city of laying out new streets, and of prescribing and limiting the powers of the Board of Supervisors and the other City and County officials.

The charter which was adopted in November, 1899, modified all this, and, among other things, created a Board of Public Works, with usual duties; but this charter, which went into effect on January 8th, 1900, made no provision for carrying to completion the proceedings relating to bond issues commenced under the Consolidation Act. The matter was carried into the courts, whose decision was to the effect that the sewer bonds already voted could not be issued. New proceedings were necessary.

During the first few years under the new charter such matters as the bonding of the city for improvements moved slowly. Each step had to be subjected to the test of the courts. It was not until 1903, therefore, that a new order was passed by the Supervisors directing the City Engineer again to prepare plans for a comprehensive system of sewers.

In accordance with this requirement, the plans of 1899, with but slight modification, were re-submitted by the writer, then City Engineer, under date of June 30th, 1903. The cost estimate was increased because it covered sewers not recommended for immediate construction 4 years before, and because there had been considerable advance in the price of materials, and particularly in wages. The estimated cost of the sewers and accessory structures, then recommended for construction, was $7 250 000.

The question relating to the issuance of sewer bonds in this amount, together with bonds for other purposes, in the aggregate about $17 000 000, was submitted to the voters in November, 1903. The bonds for sewers were again voted. But the bonds as authorized were to bear only 3½% interest. This low rate of interest coupled with the fact that the government of the city had meanwhile passed into the hands of officials who did not command public confidence, made the sale of these bonds impossible. At length further attempts to sell bonds of this issue were abandoned, and the City Engineer was called upon for a new report. This was after the great calamity of 1906. Restriction of the sewer work to the main lines of sewers was demanded, in order to keep the bond issue as low as possible. The latest recommendation, therefore, does not cover all the sub-mains that were included in the recommendations of 1903, nor even the full system of 1899.

The final submission, in May, 1908, to the voters, of the proposition to expend $4 000 000 on the sewer system again resulted favorably by a vote of about 15 to 1. Under the bond issue thus authorized the systematic construction of the projected main sewers and interceptors has now been commenced.

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AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

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THE DESIGN OF ELEVATED TANKS AND STAND-PIPES.

By C. W. Birch-Nord, Assoc. M. Am. Soc. C. E.

To be Presented May 5th, 1909.

Introduction.

It is the writer's intention to submit herewith a general specification for the design of elevated tanks and stand-pipes, which he wishes adopted by the Engineering Profession. As far as he knows this subject has never before been treated for this purpose, and it is therefore his most sincere wish that experienced engineers in this field will offer their most severe criticisms, in order to make the final revised specification a sound guide. Attention may be called to the fact that this specification has been used by the writer for some time, and has always been applied successfully.

The writer wishes to express his obligations to A. F. Reichmann, M. Am. Soc. C. E., and Mr. J. H. Hoff, for assistance in the preparation of this paper.

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

General Specifications for Stand-Pipes, Tanks and Towers.

Loads.

1.—The dead load shall consist of the weight of structural and ornamental steelwork, platforms, roof construction, piping, etc.

2.—The live load considered shall be the contents of tanks, the movable load on platforms and roofs, and the wind pressure.

3.—The weight of water shall be assumed to be 63 lb. per cu. ft., and that of crude oil 56 lb. per cu. ft., 1 cu. ft. of fluid being equal to 7.48 gal.

4.—The live loads on platforms and roofs shall be taken at 30 lb. per sq. ft., or a 200-lb. concentrated load applied at any point.

5.—The wind pressure shall be assumed at 30 lb. per sq. ft., acting in any direction. The surfaces of cylindrical tanks exposed to the wind shall be calculated at two-thirds of the diameter multiplied by the height.

6.—The movable live load on platforms and roofs shall not be considered as acting together with the wind pressure.

Unit Strains.

7.—All parts of the structure shall be proportioned so that the sum of the dead and live loads shall not cause the strains to exceed those given in Table 1.

TABLE 1.

	Pounds per square inch.	
Tension in tank plates	12 000	Net area.
Tension in other parts of structure	16 000	" "
Compression	16 000	Reduced.
Shear on rivets and pins	12 000	
Shear on bolts and field rivets	9 000	
Shear in plates	10 000	Gross section.
Bearing pressure on rivets and pins	24 000	
Bearing pressure on field rivets	18 000	
Fiber strain in pins	24 000	

8.—For compression members, the permissible unit strain of 16 000 lb. shall be reduced by the formula:

$$p = 16\,000 - 70\,\frac{l}{r}$$

where p = permissible working strain in compression, in pounds per square inch;

l = length of member, from center to center of connections, in inches;

r = least radius of gyration of section, in inches;

the ratio $\frac{l}{r}$ shall never exceed 120 for main members and 180 for struts.

9.—Stresses due to wind may be neglected if they are less than 25% of the combined dead and live loads.

10.—Unit strains in bracing and other members taking wind stresses may be increased to 20 000 lb. per sq. in., except as shown in Section 9.

11.—The pressures given in Table 2 will be permissible on foundations and bearing plates.

TABLE 2.

	Tons per square foot.
Soft clay	1
Ordinary clay	2
Dry sand and dry clay	3
Hard clay	4
Gravel and course sand	6
	Pounds per square inch.
Brickwork with cement mortar	200
Portland cement concrete	350
First-class sandstone	400
First-class limestone	500
First-class granite	600

Details of Construction.

12.—The plates forming the sides of cylindrical tanks shall be of different diameters, and shall be caulked from the inside. No foreign material shall be allowed when caulking.

In oil-tank work, both the inside and the outside of the tank shall be beveled for caulking.

13.—Joints for horizontal seams and for radial seams in the spherical bottoms of tanks shall preferably be lap joints.

14.—For vertical seams lap joints shall be used for $\frac{1}{4}$, $\frac{5}{16}$, and $\frac{3}{8}$-in. plates; double butt joints for $\frac{7}{16}$, $\frac{1}{2}$, $\frac{9}{16}$, $\frac{3}{4}$, and $\frac{13}{16}$-in. plates; and triple butt joints for $\frac{7}{8}$, $\frac{15}{16}$, and 1-in. plates.

15.—Rivets, $\frac{5}{8}$ in. in diameter, shall be used for $\frac{1}{4}$ and $\frac{5}{16}$-in. plates; rivets, $\frac{3}{4}$ in. in diameter for $\frac{3}{8}$ to $\frac{5}{8}$-in. plates, inclusive; and rivets, $\frac{7}{8}$ in. in diameter, for $\frac{1}{16}$ to 1-in. plates, inclusive.

16.—Plates more than $\frac{5}{8}$ in. thick shall be sub-punched and reamed.

17.—The minimum thickness of the plates for the cylindrical part shall be $\frac{1}{4}$ in. The thickness of the plates in spherical bottoms

shall never be less than that of the lower ring in the cylindrical part of the tank.

18.—The facilities at the plant where the material is to be fabricated will be investigated before the material is ordered.

19.—All plates shall be punched before being bevel-sheared for caulking.

20.—Radial sections of spherical bottoms shall be made in duplicates of the number of columns supporting the tank, and shall be reinforced at the lower parts, where holes are made for piping.

21.—When the center of the spherical bottom is above the point of connection with the cylindrical part of the tank, there shall be provided a girder at said point of connection to take the horizontal thrust. The horizontal girder may be made in connection with the balcony. This also applies where the tank is supported by inclined columns.

22.—The balcony around the tanks shall be 3 ft. wide, with a $\frac{1}{4}$-in. floor-plate, and shall have a suitable railing, 3 ft. 6 in. high.

23.—The upper parts of spherical bottom plates shall always be connected on the inside of the cylindrical section of the tank.

24.—In order to avoid eccentric loading on the tower columns, and local stresses in spherical bottoms, the connections between the columns and the sides of the tank shall be made in such a manner that the center of gravity of the column section intersects the center of connection between the spherical bottom and the sides of the tank. Enough rivets shall be provided above this intersection to transmit the total column load.

25.—If the tanks are supported on columns riveted directly to the sides, additional material must be provided in the tank plates riveted directly to the columns to take the shear. The shear may be taken by providing thicker tank plates or by reinforcement plates at the column connections, while bending moments shall be taken by upper and lower flange angles. Connections to columns shall be made in such a manner that the efficiency of the tank plates is not less than that of the vertical seams.

26.—For high towers, columns shall have a batter of 1 to 12. The height of the tower is understood to be the distance from the top of the masonry to the connection of the spherical bottom, or the flat bottom, with the cylindrical part of the tank.

27.—The bottom plates of stand-pipes shall be not less than $\frac{5}{16}$ in.

thick, and shall be provided with tapped holes, 1¼ in. in diameter, with screw-plugs spaced at 4-ft. centers to allow a filling of cement on top of the masonry, while the bottom part is being erected, in order to secure the proper bearing.

28.—Near the bottom of the stand-pipe there shall be provided one 12 by 18-in. manhole of elliptical shape.

29.—Near the top of each tank and stand-pipe there shall be provided one Z-bar acting as a support for the painters' trolley and for the stiffening of the tank. The section modulus of the same shall be not less than $\frac{D^2}{250}$, where D is equal to the diameter of the tank, in feet. If the upper part of the tank is held by the roof construction, this may be reduced.

30.—On large tanks, circular stiffening angles shall be provided in order to prevent the tank plates from buckling during windstorms. The distance between the angles shall be located by the following formula:

$$d = \sqrt{t}\,\frac{900}{D}$$

where, d = approximate distance between angles, in feet;
t = thickness of tank plates, in inches;
D = diameter of tank, in feet.

31.—The top of the tank will generally be covered with a conical roof of thin plates; and the pitch shall be 1 to 6. For tanks up to 22 ft. in diameter, the roof plates will be assumed to be self-supporting. If the diameter of the tank exceeds 22 ft., angle rafters shall be used to support the roof plates.

Plates of the following thicknesses will be assumed as self-supporting for various diameters:

$\frac{3}{32}$-in. plate, up to a diameter of 18 ft. 0 in.
$\frac{1}{8}$-in plate, up to a diameter of 20 ft. 0 in.
$\frac{3}{16}$-in. plate, up to a diameter of 22 ft. 0 in.

Rivets in the roof plates shall be from ¼ to $\frac{5}{16}$ in. in diameter, and shall be driven cold. These rivets need not be headed with a button set.

32.—A trap-door, 2 ft. square, shall be provided in the roof plate. Near the top of the higher tanks, a platform with a railing shall be provided, for the safety of the men operating the trap-door.

33.—An ornamental finial shall be provided at the top of the roof.

34.—A ladder, 1 ft. 3 in. wide, shall be provided from a point about 8 ft. above the foundation to the top of the tank, and also one on the inside of the tank. Each ladder shall be made of two $2\frac{1}{2}$ by $\frac{3}{8}$-in. bars with $\frac{3}{4}$-in. rungs. On large, high tanks, 30 ft. or more in diameter, a walk shall be provided from the column nearest the ladder to the expansion joint on the inlet pipe.

35.—In designing tanks, 6 in. additional height shall be allowed for over-run.

36.—The bracing in the towers shall be adjustable.

37.—The size of the anchor-bolts shall be determined by the uplift when the tank or stand-pipe is empty. The unit strains in the anchor-bolts shall not exceed 15 000 lb. per sq. in., and the minimum section shall be limited to a diameter of $1\frac{1}{4}$ in.

38.—The concrete shall be assumed to have a weight of 140 lb. per cu. ft., and shall be sufficient in quantity to take the uplift.

39.—Any parts of the tank, stand-pipe, or tower, in which difficulties may arise in field riveting, shall be assembled in the shop, and marked properly before shipment.

40.—The structural material shall conform to the "General Specifications for Steel Railroad Bridges" by the American Railway Engineering and Maintenance of Way Association.

41.—The workmanship shall be in accordance with the Manufacturers' Standard Specifications of February 6th, 1903.

42.—Before leaving the shop all work shall be painted with one coat of approved paint, excepting the laps in contact on the tankwork. All parts which will be inaccessible after erection shall be well painted. After erection, the structure shall be covered with one coat of the same paint.

43.—Three-ply frost-proof casing shall be provided, if necessary, around the inlet pipe. This casing shall be composed of two layers of 1 by $2\frac{1}{2}$-in. lumber, and each layer shall be covered with tar paper, and one outside layer of $\frac{7}{8}$ by $2\frac{1}{2}$-in. dressed and matched flooring. The lumber shall be in lengths of about 12 ft. A 1-in. air space shall be provided between the layers of lumber, and wooden rings or separators shall be nailed to them every 3 ft. The frost casing may be made square or cylindrical.

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A SYSTEM OF COST KEEPING.

By Myron S. Falk, Assoc. M. Am. Soc. C. E.

To be Presented May 5th, 1909.

The description of itemized costs of contract work always attracts some attention; unfortunately, the greatest amount of publicity has been given to the actual costs of unit items on construction work rather than to the bookkeeping methods used in obtaining those quantities.

It seems hardly necessary to call attention to the very small value that cost records may have to others than to those who gathered them and who realized and understood fully every local condition at the point of construction. There are some, however, who believe that all cost records have value, and that their publication, no matter in what form, will be a benefit. There is no doubt that the records of cost of completed work are useful to some slight extent to others than their compilers, but the greatest value of such records is derived during the time of their collection by the men who are doing the work.

By their use, a contractor can determine with reasonable accuracy the value of the work performed during any specified period; he can compare its cost with that of any other period, and he can determine

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whether the efficiency of his force is increasing or decreasing and so take the necessary steps to keep his organization in good order. It must be common experience that the superintendent at the point of construction is always an optimist; that he gauges his output for any week by the best day's work, and estimates unit costs by that gauge; his memory is short, with regard to delays and bad weather, and his estimate of the growth of work in the field is prone to error.

Cost records, therefore, have a distinct and important value to their compilers, and this paper, for which no special originality or singular merits are claimed as to the system of bookkeeping which it describes, is presented that others may describe their methods, with a view of developing some simple and uniform systems of accounting, so that a contractor may know for what purpose he is making expenditures.

The Accounting System.—The company keeps but one book, namely, a cash book; in this book there are entered on opposite pages the receipts and payments of money. The distribution of the payments to their various accounts is the basis of cost keeping.

Every payment, no matter of what kind or for what purpose, is made in the form of a blanket voucher, as shown by Figs. 1 and 2; each payment as made is numbered consecutively in the cash book, and this number appears on the voucher and on every bill, order, or blank which concerns in any way the payment in question; the tracing of orders, material received, etc., is checked by noting whether a voucher number is marked upon it.

On the back of the voucher (in the proper blank space), after the bill has beeen receipted, there is written the account or item against which the payment is to be charged. This payment may be charged against any item of any particular contract, or against any general or overhead expense account, such as traveling, rent, or telephoning. The charge on the back of this voucher is entered in the cost-keeping ledger, which is a loose-leaf book, a sample page of which is shown by Fig. 3. Each page represents one item, and particular attention is called to the left half of the page, which represents expenditures. These expenditures are itemized in two groups and a "Total"; the first column, headed "Labor," represents labor charges; the second column shows any and every other expense chargeable to the main item; and the third, headed "Total," shows the sum of the preceding two, whenever desired. It is evident that this page is the key to the

Co., *New York*_____*190*

*To*_____*Dr.*

As per bill attached.

PLEASE DATE AND SIGN THIS AND RETURN PROMPTLY,
TOGETHER WITH ATTACHED BILLS, ALSO RECEIPTED.

Correct as to calculation,

_____

Checked and approved,

_____

*Received*_____*190*

_____Dollars

in payment of above account.

_____

_____

$_____

PLEASE FOLD THIS VOUCHER BUT ONCE.
PLEASE RETURN THIS VOUCHER RECEIPTED IMMEDIATELY.

*Voucher No.*_____

_____Co.

*Name*_____

*Date*_____*190*

$_____

CHARGEABLE TO

General Expense
Salary
Sundry
Traveling
Office Equipment

Contract at _____
Plant
Engineering
Labor
Supplies and Materials
Sundry

Total

Signature examined and voucher filed by _____

PLEASE FOLD THIS VOUCHER BUT ONCE, AND ON THE LINE.

Standard Size, 7 by 8⅛ in.

FIGS. 1 AND 2.

Sheet No. 3

..... CONSTRUCTION CO. CONTRACT .....

CHARGE TO ITEM NO. 3. Eight-Axed Granite Masonry

DATE		VOUCHER NO.	EXPENDITURES	LABOR					TOTAL	DATE		RECEIPTS	TOTAL
Mar	11		Unloading Lighter Pier III		1	7	50						
"	12		loading for Pier II Cover A		1	7	50						
"	13		Rain all day			6	45						
"	14		laid Pier III A 387 cu. ft.		3	6	32						
"	15		" " B.C. 237 " "		2	8	65						
"	19		Rain and Snow			7	20						
"	20	624	F. Baldwin Manilarope					3	1 40				
"	"	630	Red Star Towing Co. towing					3	0 —				
"	"	631	O. T. Stephens — Coal etc					1	3 1 25				

Standard Size 9 in. wide and 12 in. high.

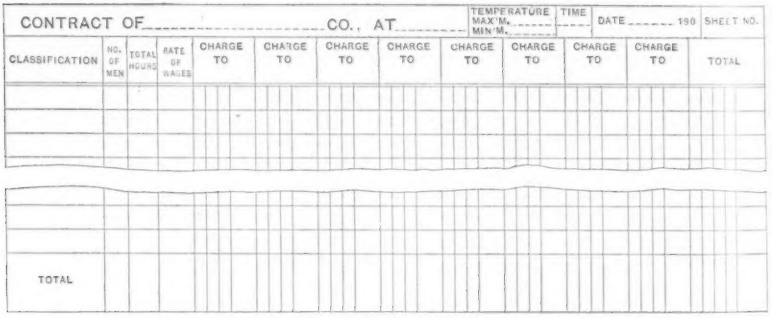
FIG. 3.

Foreman.....
Timekeeper.....
Date.....

NAME	HOURS	RATE	NAME	HOURS	RATE

entire system; the cost of labor is maintained entirely distinct from every other charge, and that cost is precisely the one most desired, as other costs, for plant, raw material, or anything else, can, as a rule, be determined or estimated without essential error.

Before describing the entries in this book, it is necessary to state that as soon as the company receives a contract, it prepares a list showing the items under which all expenses are to be subdivided. If the contract is based on unit prices, these items include the various units of the contract; for example, piles, or grillage timber, or waterproofing, etc., as well as items for plant erection, or plant removal, or plant maintenance, which cover expenses that cannot be charged against any one unit until the completion of the work. Again, many of the unit items of the contract are in turn subdivided.



Standard Size, 5 by 8 in.

FIG. 6.

As soon as a blanket voucher is receipted and returned to the company, it is charged against the proper item in the cost-keeping ledger, and is then filed, the only exception being in the case of vouchers for pay-rolls. As one pay-roll voucher covers either one or two weeks' work, it obviously cannot be entered in the ledger until it is subdivided into unit charges. This is performed in the field by the timekeeper, who uses the cards shown in Figs. 4, 5 and 6. Fig. 4 shows the pay-roll card for each day; each man's name or number, together with the amount earned by him, is entered upon this, and, on the reverse side (Fig. 5) is tabulated all the labor for the day. This tabulation is then carried forward to the card shown by Fig. 6, and is there itemized and charged against the various items of the contract. If the pay-roll is met weekly, seven of these cards

CONTRACT OF ____ Co., AT ____ TEMPERATURE MAX'M ____ MIN'M ____ TIME ____ DATE 8/29/08 190 SHEET NO. ____

CLASSIFICATION	NO. OF MEN	TOTAL HOURS	RATE OF WAGES	CHARGE TO Gravel Side Walk	CHARGE TO 2" Pipe and Railing	CHARGE TO Asphalt Blocks	CHARGE TO Plant Const	CHARGE TO maintenance	CHARGE TO #2 Concrete	CHARGE TO Curbing	CHARGE TO Ashlar Masonry	TOTAL
Foreman	1	8				(11)	5 00				(2)	5 00
Laborers	51	405½	20	5 60	16 80	33 10	(32)		25 60	(32)		81 10
" 2	4	28¾		(34)	(42)	3 34	2 06				1 27	6 67
" 3	2	16	35-			2 80		(45c.)	2 80			5 60
Masons	2	12		1 75	1 75				(12)		2 75	6 25
Dogs	8	64	4 3¾				14 00	14 00				28 00
St Engines	2	16				4 20	3 75					7 75
Sr. Cutters	2	8	56¼							4 50		4 50
Curb Setters	2	8								3 75		3 75
Boy & Horse	1	8				3 60						3 60
Oscars "	1	8				2 00						2 00
Watchman	1	-		25-	25-	25-	25-	25-	25-	25-	25-	2 00
Timekeeper	1	-			25-	25-	25-	25-	25-	25-		1 50
Engines	1	-		75-	1 00	50	50	1 00	1 00	50	25- (01)	5 00
TOTAL	79			7 85	20 05	50 04	25 81	15 50	24 90	9 25	4 52	$162.92

Laid 30 Sq yds. Dolf 2" Pipe and 6 railing Sundry repairs Unloaded 23,500 asphalt Blocks East side Setting up derrick Pier 4 - Moved Stock House - Mixed 61⅓ Sq. yds. of 1-2-4 Concrete Dolf 69 - Cut 15-16" Planning and pointing Pier

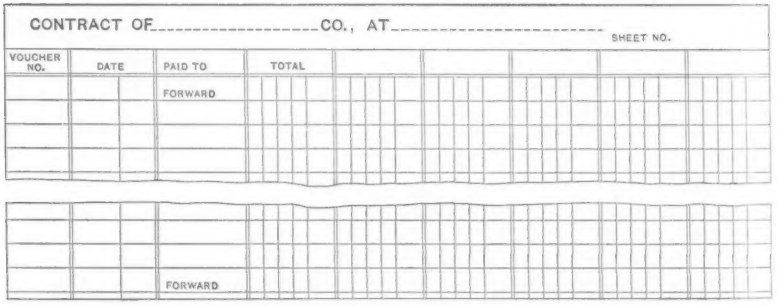
Standard Size, 5 by 8 in.

FIG. 7.

Papers.]

must equal the total shown on the blanket voucher of the cash book. There is also written on the itemized pay-roll cost card (Fig. 6) the amount of work accomplished on each item; the quantities are obtained by the timekeeper each day from the various foremen without any difficulty and with comparatively small error. As an example, Fig. 7 is reproduced; this is an actual card turned in by the timekeeper. As soon as the pay-roll and blanket voucher are made up, the items on the cards representing that roll are entered in the cost-keeping ledger, as shown by Fig. 3.

The items in this book are totaled in pencil, at intervals of a few days or a week, in order to note the progress and cost of the work; each page is permanently totaled and checked in ink on the first day of each month, and checked against a supplementary card-system, which



Standard Size, 5 by 8 in.

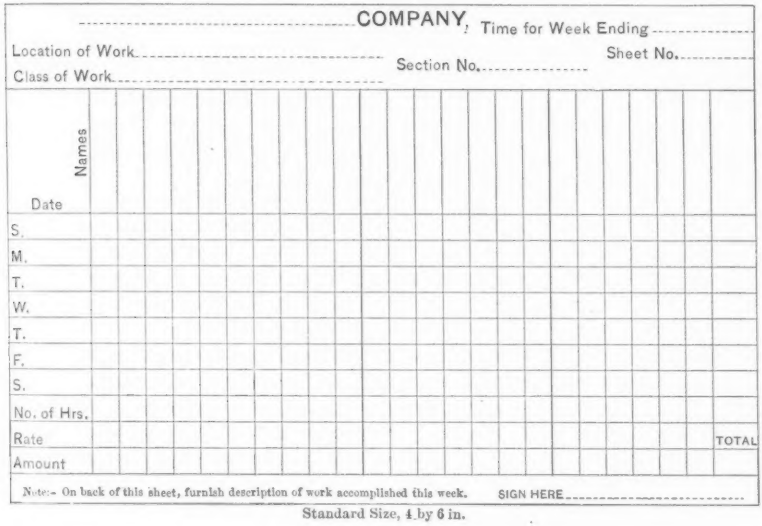
FIG. 8.

records the total cost of any contract, in the condensed form shown by Fig. 8. On this card, each blanket voucher is also entered as soon as filled out, and is itemized into one of five general headings; usually these headings are (1) Labor, (2) Plant, (3) Material Remaining in Work, (4) Other Material, and (5) Sundries, there being five columns on each page. This supplementary card ledger is extremely useful in showing at any time the general divisions of expense.

To complete the description of the system, the card illustrated by Fig. 9 is shown. This card is the summary of seven of the daily cards shown by Fig. 4, and is used in making up the weekly pay-rolls.

It has been found that the cards shown by Figs. 4 and 6 might be combined in one card in order to save the time of the timekeeper, but the sheet resulting from the combination has been found

to be too bulky to be used on a stiff card, in which form all the records of the company are now kept, and, rather than use thin paper records, the present card system is maintained.



Standard Size, 4 by 6 in.

FIG. 9.

As used by the company, the foregoing system has furnished the office force precisely the information which it needed at any time in order to trace loss of efficiency in the field; and the records of the completed work, particularly in respect to the item of labor, are in the form needed when other and similar work is estimated. The system described is that used by the Godwin Construction Co., of New York.

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NICKEL-STEEL EYE-BARS FOR BLACKWELL'S ISLAND BRIDGE.*

By William R. Webster, M. Am. Soc. C. E.

Having obtained some very interesting results, in connection with the preliminary experimental tests of carbon and nickel-steel built-up riveted eye-bars and forged nickel-steel eye-bars, and in the tests of the eye-bars used in the Blackwell's Island Bridge, it seemed to the writer that it would be desirable to give a summary of this work, and to place on file in the Library of the Society, a report, with tests, etc., where it would be accessible to those interested.

The eye-bars used in the structure were the largest ever manufactured, some of them being 16 by 2⅛ in., with heads 37½ in. in diameter, with 16-in. pins, and more than 64 ft. from center to center of pin holes. Special machinery had to be made for upsetting and rolling the eye-bar heads, and a testing machine of 2 000 tons capacity had to be designed and built to break them.

Nickel steel had been used for years in forgings, with special heat treatment. The best results were obtained by oil-tempering and annealing. When it was proposed to use nickel steel for eye-bars, it

*Additional data on this subject are filed in the Library of the Society, where they may be examined by any one who is interested.

This paper will not be presented at any meeting, but written communications on the subject are invited for publication with it in *Transactions*.

was apparent that the hot bars would be distorted in handling during the process of oil-tempering and annealing, and it became necessary to learn what results could be obtained by modifying the ordinary methods of annealing carbon-steel eye-bars.

The first nickel-steel eye-bars were manufactured by The American Bridge Company at Edge Moor, and were tested at Pencoyd, in 1902. There were twenty-one of these bars, 6, 8 and 10-in., of basic open-hearth steel containing 3.26% of nickel. Records of the tests of these bars are filed in the Library of the Society, and a summary is given in Table 1. The elastic limits were determined by the halt in the mercury column, in the usual way.

In February, 1903, the Commissioner of Bridges, of the City of New York, requested the writer to make a series of experimental tests of nickel-steel eye-bars, and prepare specifications for steel of different qualities required for the construction of the Blackwell's Island and Manhattan Bridges. Eight bars of basic open-hearth steel, containing 3.24% of nickel, were made and tested at Pencoyd. Records of these tests, with the specifications prepared at that time, are filed in the Library of the Society, and a summary is given in Table 2. The elastic limits of this lot of eye-bars were taken by scribe marks of a finely pointed beam compass on a base length of 10 ft. A summary of the specification referred to above, and all other specifications referred to in this paper, are given in Table 3.

When submitting these specifications the writer stated:

"In preparing these specifications all the tests that had been made at Pencoyd, and other information at hand, were considered. The requirements may seem a little severe, but if proper care is taken in the manufacture of the steel, can all be met.

"In order to insure greater care in the manufacture and uniformity in the steel, high and low limits are specified for ultimate strength in all cases. The leeway of 15 000 lb. seems to be wide enough, and should not have to be increased. If the upper limit is omitted, the same amount of care will not be exercised in rolling the steel, very little attention will be paid to the finishing temperature, higher carbons will probably be used, and reliance will be placed on the results of the annealed test pieces to accept the steel.

"Tension and bending tests are called for on the bars as rolled without annealing. These are additional checks, to insure proper care being taken in the manufacture, and prevent brittle steel being used. If this is not done, and annealed test pieces are relied on alone to

TABLE 1.—Comparison of Specimen Tests of Nickel-Steel Bars, Unannealed and Annealed, with Eye-Bar Tests.

Bars manufactured by Carnegie Steel Co.
Eye-bars manufactured and tested by American Bridge Co.

Analysis: All from Heat No. 10450 — C. 0.21, P. 0.007, S. 0.016, Mn. 0.53, Si. 0.011, Ni. 3.265.

Nickel Steel.		Specimen Tests.				Full-Sized Eye-Bar Tests.				Fracture.
		Unannealed.		Annealed.						
Mark of Bar.	Size of Bar.	Elastic limit.	Ultimate strength.	Elastic limit.	Ultimate strength.	Elastic limit.	Ultimate strength.	Percentage of elongation in 10 ft.	Percentage of reduction.	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
A 1 *x*	6 by 1 in.	47 815	79 510	49 120	77 140	46 230	74 450	13.2	42.40	Silky, Angular.
A 2 *x*	"	48 190	80 300	50 895	75 890	46 300	74 100	16.2	49.80	" ½ Cup.
A 3 *x*	"	47 330	79 960	47 127	73 320	43 100	69 000	17.7	49.60	" " "
A 4 *x*	"	50 655	79 300	44 460	72 760	45 510	70 420	16.2	54.80	" " "
B 1 *x*	6 by 2 in.	46 963	76 930	48 945	76 920	47 300	72 200	16.3	42.90	" Cup.
B 2 *x*	"	46 615	78 880	48 405	78 260	46 260	74 290	17.4	46.90	" "
B 3 *x*	"	48 220	75 840	47 365	70 060	44 040	68 200	15.3	53.10	" ½ Cup.
B 4 *x*	"	50 335	78 420	48 515	74 170	43 120	69 340	12.7	44.23	" Angular.
B 5 *x*	"	48 215	76 360	51 663	78 790	44 860	72 400	12.1	41.10	" "
C 1 *x*	8 by 1½ in.	.....	.....	45 805	78 190	42 500	73 600	17.1	41.20	" ½ Cup.
C 2 *x*	"	47 813	76 970	47 730	70 240	40 420	66 150	20.8	57.50	" Cup.
C 3 *x*	"	48 410	79 260	48 620	78 480	43 510	74 720	14.5	51.46	" "
C 4 *x*	"	46 630	77 710	48 005	74 620	48 220	77 080	18.3	48.01	" ½ Cup.
C 5 *x*	"	47 460	79 170	48 650	74 090	41 450	74 450	16.8	47.44	" Angular.
C 6 *x*	"	48 590	79 730	47 146	73 740	43 730	73 620	17.1	49.69	" Cup.
E 1 *x*	10 by 1¼ in	47 840	79 110	48 740	79 240	46 320	72 920	13.7	47.00	" Angular.
E 2 *x*	"	48 650	79 660	48 220	73 440	41 630	69 810	15.3	38.40	" Cup.
E 3 *x*	"	47 130	74 270	48 170	78 110	46 480	76 540	15.5	42.20	" "
E 4 *x*	"	50 020	78 930	47 210	77 380	46 070	74 540	13.1	43.70	" "
E 5 *x*	"	48 320	80 320	47 970	75 710	42 960	73 290	17.1	51.70	" "
E 6 *x*	"	47 620	79 630	48 110	78 070	44 530	75 410	15.1	40.00	" Angular.

Elastic limit of specimen tests taken by drop of beam; of full-sized eye-bar tests by halt in gauge. The figures in Columns 5 and 6 are the results of specimen tests of annealed bars.

TABLE 2.—Comparison of Specimen Tests of Nickel-Steel Bars, Unannealed and Annealed, with Eye-Bar Tests.

Bars manufactured by Carnegie Steel Co.
Eye-bars manufactured and tested at Pencoyd.

All from the same heat.
Analysis: C. 0.35, S. 0.022, Phos. 0.02, Mn. 0.68, Ni. 3.24.
Philadelphia, April 4th, 1903.

Nickel Steel.		Specimen Tests.				Full-sized Eye-bar Tests.				Fracture.
		Unannealed.		Annealed.						
Mark of Bar.	Size of bar.	Elastic limit.	Ultimate strength.	Elastic limit.	Ultimate strength.	Elastic limit.	Ultimate strength.	Percentage of Elongation in 18 ft.	Percentage of Reduction.	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
H 1 *x*	6 by 2 in.	52 800	90 650	47 040	79 610	47 820	85 000	9.17	42.47	Silky, ¼ Cup, 5% Crystalline.
"	"	51 020	88 370	49 320	76 900					
H 2 *x*	6 by 2 in.	50 740	101 000	48 190	88 600	46 350	83 780	10.76	52.17	Silky, ½ Cup.
"	"	45 190	98 600	47 620	86 050					
H 3 *x*	6 by 2 in.	30 300	98 610	49 090	86 600	48 580	87 190	9.77	38.80	40% Crystalline, Irregular.
"	"	54 250	102 600	47 990	91 450					
H 4 *x*	6 by 2 in.	45 690	106 290	48 670	86 600	48 230	86 130	9.50	41.90	¾ Cup, 25% Crystalline.
"	"	49 780	101 920	48 360	88 850					
H 5 *x*	6 by 2 in.	49 960	102 460	51 760	90 260	48 380	86 490	9.50	44.20	Irregular, 30% Crystalline.
"	"	51 280	99 400	50 520	86 750					
J 1 *x*	6 by 1½ in.	49 950	95 150	46 240	83 140	49 350	86 240	10.44	45.75	Cup, 15% Crystalline.
"	"	50 520	103 420	44 800	86 780					
J 2 *x*	6 by 1½ in.	49 080	96 400	47 110	80 000	49 300	85 490	10.27	44.00	½ Cup, Crystalline, Specks on both edges.
"	"	49 450	99 670	43 180	98 800					
J 3 *x*	6 by 1½ in.	49 190	91 780	44 150	83 810	49 360	86 160	8.44	42.74	¾ Cup, 10% Crystalline on one edge.
"	"	53 390	111 120	44 780	83 600					

Elastic limits taken with dividers; 8-in. gauge length specimens; 10-ft. eye-bars.
The figures in Columns 5 and 6 are the results of specimen tests of annealed bars.

TABLE 1.—Comparison of Specimen Tests of Nickel-Steel Bars, Unannealed and Annealed, with

TABLE 3.—SUMMARY OF SPECIFICA-

Specifications.	Unannealed Specimens.					Annealed	
	Ultimate strength.	Elastic limit.	Percentage of elongation in 8 in.	Percentage of reduction.	Bends.	Ultimate strength.	Elastic limit.
William R. Webster's original specifications	90 000 to 105 000	55 000	18	35	180° 37'	85 000 min.	52 000
Blackwell's Island Bridge original printed specifications	90 000 to 105 000	52 000	do.	do.	do.	85 000 to 100 000	50 000
Blackwell's Island Bridge contract specifications	100 000 min.	55 000	1 600 000 ultimate.	to be recorded.	do.	85 000 min.	48 000
Modifications:* Lot or item tests	95 000 to 110 000	do.	do.	do.	do.	do.	do.
Test each heat with high tensile strength	113 000 min.	do.	do.	do.	do.	do.	do.
For bars which break in head	95 000 min.	do.	do.	do.	do.	do.	do.
Conclusions Requirements that can be met as proven by tests of bars for structure.	95 000 to 110 000	do.	do.	35	do.	90 000 min.	52 000

* Tensile in full-sized tests reduced from 85 000 lb. down to 80 000 lb., all other requirements to remain as specified.

Where only one amount is given it means minimum.

check the quality of the material, there is nothing to insure the finished eye-bars being annealed in the same manner and similarly restored. Whereas, starting with a known good material, the annealing will further improve it."

These specifications for nickel-steel eye-bars were submitted to the Board of Engineers, consisting of Messrs. Theodore Cooper, Henry W. Hodge, Mansfield Merriman, George S. Morison, and C. C. Schneider, Members, Am. Soc. C. E., appointed to consider an eye-bar cable for the Manhattan Bridge.

They reported that a nickel-steel forged eye-bar cable, made to these specifications, would be satisfactory to them, except that they considered the elastic limit of 50 000 lb. per sq. in. a little too high,

TIONS FOR NICKEL-STEEL EYE-BARS.

Specimens.			Full-Sized Eye-Bars.					Chemistry.			
Percentage of elongation in 8 in.	Percentage of reduction.	Bends.	Ultimate strength.	Elastic limit.	Percentage of elongation in 18 ft.	Percentage of reduction.	Location of fracture.	Phos.		S.	Ni.
								Acid.	Basic.		
20	40	180° 2T.	85 000	50 000	9	40	In body.	0.06	0.04	0.05	3.25 to 3.50
20	40	do.	do.	48 000	do.	40	do.	do.	do.	do.	3.50
1 600 000 ultimate.	to be recorded.	do.	do.	do.	do.	to be recorded.	do.	do.	do.	do.	do.
do.	do.	do.	85 000 and 80 000	do.	do.	do.	do.	do.	do.	do.	do.
do.	do.	do.	85 000	do.	do.	do.	do.	do.	do.	do.	do.
do.	do.	do.	do.	51 000	6	do.	In head.	do.	do.	do.	do.
1 800 000 ultimate.	40	do.	do.	50 000	9	35	In body.	do.	do.	do.	do.

All bending tests are full thickness, and not less than 4 in. wide.
Elastic limits of full-sized tests to be taken by extensometer on base length of 10 ft., a permanent set of 0.025 in. to be considered the elastic limit.

and suggested modifying it to 48 000 lb.; also, that any visible set in the eye-bar should be taken as the elastic limit, using the best means to determine what constituted a visible set—a magnifying glass if necessary.

These specifications were used when the city first asked for bids on the Blackwell's Island Bridge, but the contract was not awarded at that time.

One of the manufacturers questioned the narrow leeway of 15 000 lb., on account of the influence of segregation on the ultimate strength in as large an ingot as would be required to roll the heavy bars. In order to determine what differences would be found, he rolled two bars, from two heats of nickel steel, 18 by 2 in. and 18 by 2¼ in., each more than 65 ft. long. Tests were taken from each end of these

bars to represent the top and bottom of the ingots, with the following results:

18 by 2-in. plate.....	test cut from	top,	96 940	lb.	tensile.
.....	" " "	bottom,	89 000	"	"
18 by $2\frac{1}{4}$-in. plate.....	" " "	top,	82 040	"	"
.....	" " "	bottom,	80 750	"	"

These results were so satisfactory that he agreed that the leeway of 15 000 lb. was sufficient.*

When the city asked for bids the second time, the specifications were changed materially, and a minimum limit of 100 000 lb. tensile, with no maximum limit, was specified.

After the contract for the bridge had been awarded, it was suggested to use built-up riveted eye-bars in place of the forged eye-bars called for in the contract. The writer, having been appointed Inspecting Engineer, was instructed to make a large series of tests, on bars of this kind, of both high-carbon steel and nickel steel, in order to determine their relative merits.

There were tested twenty-two high-carbon steel built-up riveted eye-bars, of the following dimensions:

8	Eye-bars,	24	by 1	in.,	with	8-in.	pins.
4	"	24	" $1\frac{1}{8}$	"	"	8 "	"
2	"	24	" $\frac{3}{4}$	"	"	8 "	"
2	"	31	" $\frac{3}{4}$	"	"	12 "	"
6	"	32	" $\frac{3}{4}$	"	"	12 "	"

and twenty-five nickel-steel built-up riveted eye-bars of the following dimensions:

8	Eye-bars,	24	by 1	in.,	with	8-in.	pins.
10	"	24	" $\frac{3}{4}$	"	"	8 "	"
7	"	32	" $\frac{3}{4}$	"	"	12 "	"

All the riveted bars had one pin-plate on each side, and the rivets were countersunk. All holes were drilled from the solid, and all sheared edges planed; most of the bars had two lines of $\frac{15}{16}$-in. drilled holes along the body. The center-to-center distances of pin holes were 23 ft. 6 in. and 25 ft.†

From six to eight tension and bending tests were made on specimens from each plate, of both high-carbon and nickel steel, used in these

*A copy of all tests is filed in the Library of the Society.

†Fully detailed drawings of these bars have been filed in the Library of the Society.

built-up eye-bars. Special care was taken in the heating and rolling of these steels.*

In order to determine the elastic limits as accurately as possible, the writer had a special extensometer made. It multiplied ten times, could be relied on to about 0.005 in., and was easily attached to the bar. Fig. 1 shows this instrument and the method of using it.

None of the riveted eye-bars gave satisfactory results, as shown by the detailed reports of the tests.† The loss of strength in the carbon-steel bars, from the average of the specimen tests, was from 11 to 41% on the net area, and from 20 to 46% on the gross area; and, in the nickel-steel bars, from 7 to 13% on the net area, and from 13 to 22% on the gross area.‡

This loss of strength was greater than would be expected from the results of tension tests on small riveted joints or on small narrow bars with drilled holes in them, and is no doubt due to the wide thin plate tearing when tested in tension. Particular attention is called to this, as it has often been suggested to make very large eye-bars from wide plates by machining them narrower in the center in order to prevent the loss of strength which occurs in forged eye-bars due to annealing. Before large bars of this kind are used, a complete series of tests should be made, in order to show what the actual loss in strength would be in a wide, thin plate, as compared with a forged eye-bar of half its width and double its thickness made from steel showing the same tensile strength in the specimen tests.

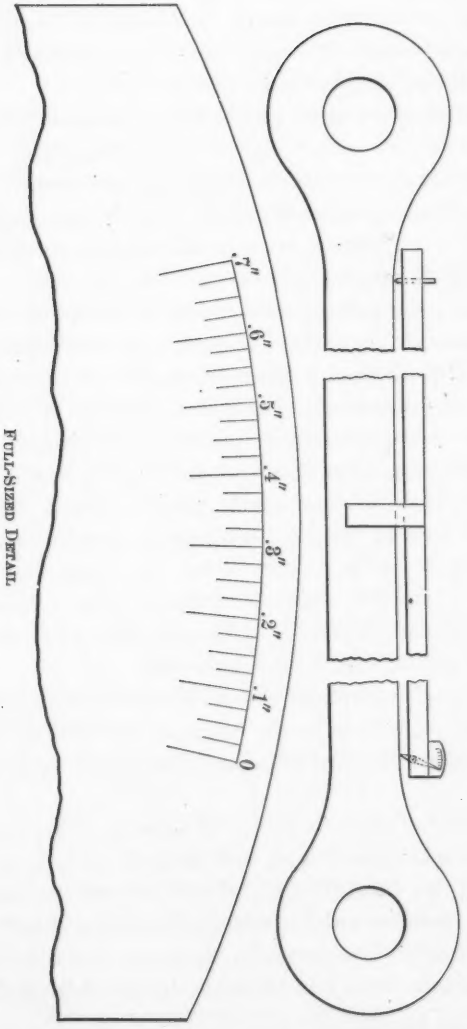
Fig. 2 illustrates an end of one of the 24-in. riveted bars, and shows the transmission of the strains by the rivets, as indicated by the cracking of the mill scale on the pin-plates, in a remarkably uniform manner.

All the riveted carbon-steel eye-bars broke off square, with granular fracture, little or no reduction of area, and elongations from 0.06 to 3.21% in the body of the bar. The nickel-steel riveted eye-bars, made of the softer heat of steel, containing 0.30% of carbon and 3.72% of nickel, broke with from 50 to 85% granular fractures, small reductions of area, and elongations from 2.17 to 3.11% in the body of the

*A report of these tests, together with a diagram showing the location of the test pieces in the plate, is filed in the Library of the Society.

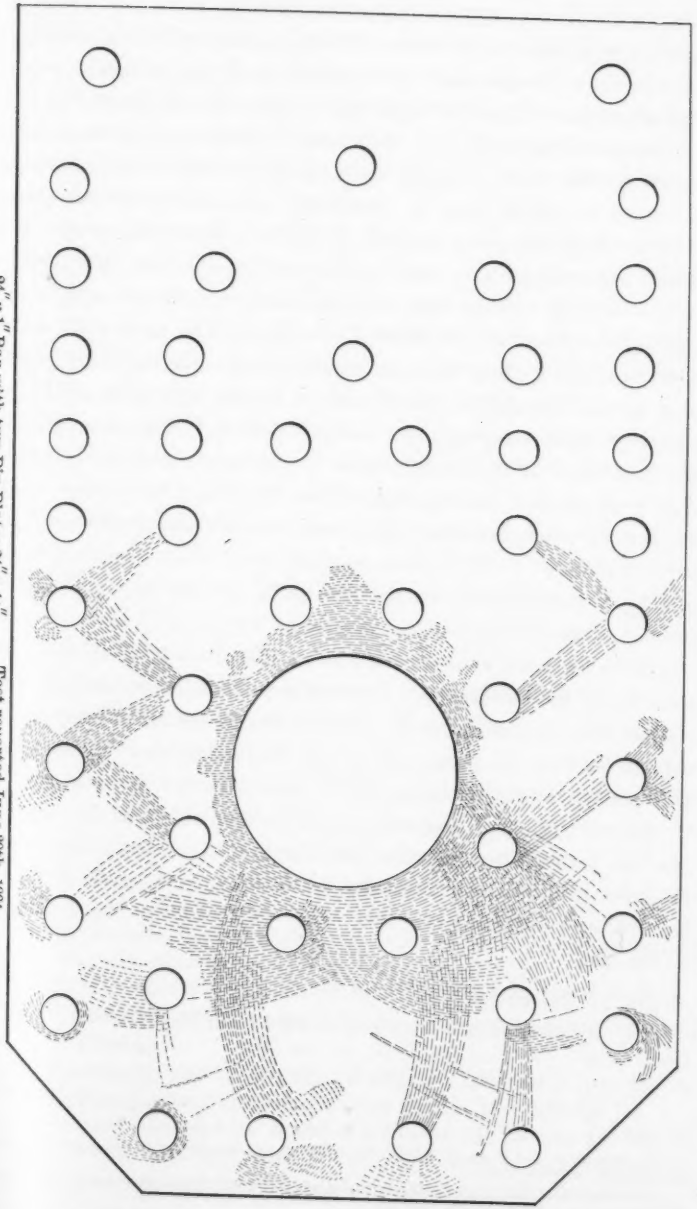
†Filed in the Library of the Society.

‡Full details of all tests, with extensometer readings, have been filed in the Library of the Society.



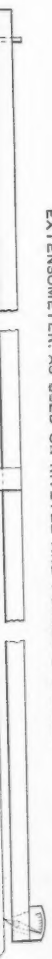
FULL-SIZED DETAIL.
FIG. 1.

EXTENSOMETER, AS USED ON RIVETED AND FORGED EYE-BARS.



24″ x 1″ Bar with two Pin-Plates 24″ x ½″. Test reported June 30th, 1904.

FIG. 2.



EXTENSOMETER AS USED ON RIVETED AND FORGED EYE-BARS.

bar; and all the harder steels, containing 0.477% of carbon and 3.56% of nickel and 0.424% of carbon and 3.57% of nickel, broke off square, with granular fractures, little or no reductions of area, and elongations from 0.78 to 2.39% in the body of the bar.*

Before these full-sized tests were completed, instructions were issued to the writer to make tests on nickel-steel forged eye-bars and to report on which bars he considered best suited for the structure. Twelve 14 by 2-in. bars, and six 16 by 2-in. bars were tested. Half of these bars were of acid open-hearth steel containing 0.33% of carbon and 3.72% of nickel; and half of basic open-hearth steel containing 0.36% of carbon and 3.40% of nickel. The same care was exercised in rolling these bars as in rolling the former bars. The eye-bars were all forged by The American Bridge Company, at Ambridge, Pa., and were tested at the works of The Phœnix Iron Company, Phœnixville, Pa., but some of the larger bars could not be broken as they were beyond the capacity of the machine.

Tension and bending tests were made from each end of each bar rolled.†

The writer reported on the results of all the tests made, advising the use of forged nickel-steel eye-bars.‡

Table 1 gives a summary of the results of all tests made on nickel-steel bars; the comparisons there given are between the net sections of the riveted bars and the forged bars. Table 2 gives the comparison between the gross section and the forged bars. For comparison, the average results of the specimen tests are given in each case.

Before any work was started on rolling the flats or forging the eye-bars for the structure, the following full-sized tests were called for, in order to test the uniformity of the material, methods of manufacture, and whether the upset head would develop the full strength of the bar:

Four heats of steel to be rolled:

3	full-sized	eye-bars	to	be	tested	from	the first heat
3	"	"	"	"	"	"	second heat
2	"	"	"	"	"	"	third heat
2	"	"	"	"	"	"	fourth heat

*A blue-print and a report, giving the location and character of the fractures, is filed in the Library of the Society.

†Full details of all specimen tests and full-sized tests, showing extensometer readings, are filed in the Library of the Society.

‡Report filed in the Library of the Society.

Owing to the delay in completing the upsetting machine and the testing machine, however, about 2 000 tons of nickel-steel flats for eye-bars were rolled before these preliminary tests were made. As the contract specifications did not give any upper limit in tensile strength, the 2 000 tons of flats really consisted of two grades of steel, as the ultimate strengths in the sample tests ranged from 100 000 to as high as 126 000 lb. per sq. in.

The results of the ten full-sized tests of eye-bars from the first four heats were so irregular that they were considered experimental tests, and the manufacturer decided to test all the eye-bars which had been forged up to that time. These bars were annealed under varying conditions, using a pyrometer to determine the annealing temperatures, and were tested to destruction. Additional bars were also forged, annealed, and tested by the manufacturer before any eye-bars were submitted for test and inspection for the structure. In all, there were seventy-three of such bars tested.

In this preliminary work it was found that the results of tests on full-sized eye-bars, made from heats of steel of more than 115 000 lb. per sq. in. tensile in the sample, were so irregular that it was decided to make a full-sized test on an eye-bar from each of such heats of the 2 000 tons already rolled, and on all steel to be rolled, to make a full-sized test on each heat of more than 110 000 lb. per sq. in. tensile in the sample test, if the annealed sample test showed more than 100 000 lb. per sq. in.

As the work progressed, the following additional modifications were found advisable, in order to secure satisfactory eye-bars for the structure and prevent the use of brittle steel:*

Ultimate strength in unannealed specimen tests reduced to 95 000 lb. per sq. in.; all other provisions to stand as specified.

Ultimate strength in full-sized tests reduced from 85 000 to 80 000 lb. per sq. in.; all other provisions to stand as specified.

Eye-bars of heats of from 95 000 to 110 000 lb. per sq. in. in the unannealed specimen to be tested by lots or items, and 3% of such bars to be tested.

Test by heats all eye-bars made of steel of 113 000 lb. per sq. in. tensile, and over, if the annealed specimen test is over 100 000 lb.

Test by heats all eye-bars made of steel of 111 000 to 113 000 lb. per sq. in. tensile in the specimen test when carbon and manganese are very high and the annealed specimen test is over 100 000 lb.

*A report on these preliminary tests of 16-in. bars, and modifications desired, is filed in the Library of the Society.

Desired: Carbon not over 0.40, if manganese is 0.80 or over.
Desired: Carbon not over 0.45, if manganese is 0.70 or over.

In all the preliminary experimental eye-bars and those for the structure, the elastic limits were taken by the extensometer. Tests of eye-bars for the structure were made. The extensometer readings,* are grouped as follows:

Bars that met the requirements of the specifications;
†Bars that broke in the head, but were accepted under a modification of the specifications;
Bars that failed in elastic limit;
Bars from special high heats which broke in the head, with low elongation.

For comparison, some tests of carbon-steel eye-bars, showing the extensometer readings, are also filed in the Library of the Society.

All the nickel-steel eye-bars were tested on The American Bridge Company's large testing machine at their Ambridge Plant. The results on this machine are a little lower than those on the Government machine at the Watertown Arsenal.†

The clause in the printed specification covering the annealing of eye-bars was not satisfactory for nickel steel, and the manufacturer determined the proper annealing temperature in the series of full-sized eye-bar tests referred to—the ten tests from the first four heats and the other full-sized tests made at that time. The pyrometer was used to determine temperatures in annealing all bars for the structure.

The bending tests on the bars showed that the wide full-thickness bends on unannealed specimens (180° $3T$) are a good check on the methods of rolling, and the full-sized bends of the pieces annealed with the eye-bars, a good check on the annealing.

The results of all the tests prove conclusively that a nickel steel of from 95 000 to 110 000 lb. per sq. in., tensile, in the unannealed specimen, and of 90 000 lb. per sq. in., minimum tensile, in the annealed specimen, would give eye-bars with a minimum elastic limit of 50 000 lb. per sq. in. in full-sized test and guard against brittle steel. The desired ultimate for full-sized tests was 90 000 lb. per sq. in., but 85 000 lb. would be accepted provided the eye-bars would show a minimum elastic limit of 50 000 lb. per sq. in.

*Filed in the Library of the Society.

†The Commissioner of Bridges modified the specifications, so that bars which broke in the head but developed an elastic limit of 51 000 lb. and an ultimate strength of 85 000 lb., with stretch of 6% in 18 ft., would be accepted.

‡The report of the standardization of the Ambridge machine is filed in the Library of the Society.

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AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

THE OPERATION OF PASSENGER ELEVATORS.

Discussion.*

By Messrs. F. Lavis, Robert Brewster Stanton, and William J. Boucher.

F. Lavis, M. Am. Soc. C. E.—It might be interesting if Mr. Bolton would state why it is necessary to operate elevators on a schedule. The service does not seem to be comparable to railroad service, because, carrying out the simile, each elevator has its own track and its own station, and apparently each could be run entirely independently of all the others. Mr. Bolton gave an illustration of a case where there was a battery of six elevators, one having a much smaller door than the others and, consequently, taking longer to load and unload. He stated that the round-trip time of this slow elevator controlled the round-trip time of the other five. It would almost seem that this one might have been operated independently and the other five run on their own schedule, if a schedule was required. Mr. Lavis.

At times the speaker has occasion to visit the Park Row Building, where there is a large battery of elevators, and he has often noticed as many as three elevators on the ground floor at once, the waiting passengers being only allowed to enter one; when that was filled, those remaining might enter the second, and when that was filled, and started, the passengers still remaining might enter the third, by which time there were probably other cars waiting, and, at first sight, it would seem that time was being wasted, and that the cars might be

*This discussion (of the paper by Reginald Pelham Bolton, M. Am. Soc. C. E., printed in *Proceedings* for December, 1908), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

Mr. Lavis. started and make their round trip each as quickly as possible and independently of the others.

No doubt there are very good reasons for the operation of elevators in rotation and as nearly as possible on schedule time, and of course regularity, as a general principle, is generally conceded to be a good thing; but it is not always safe for engineers to figure on generalities, and the speaker believes some definite reasons for the necessity of schedule operation would be of interest.

Mr. Stanton. ROBERT BREWSTER STANTON, M. AM. SOC. C. E.—To most engineers who have not made the question a study, it is, perhaps, somewhat surprising that the general problems of railway transportation enter so extensively into "The Operation of Passenger Elevators." The author has stated that different classes of passengers—the fat man, the lean man, the man who rushes in or out before the gates are fairly open, and the man who never "steps lively," as well as the woman with the hat 3 ft. in diameter—all affect the successful operation of passenger elevators, especially in tall office buildings.

It may not be out of place to ask the author whether, in his calculations, the question of classification of passengers has been taken into account in order to determine its effect upon the successful, and as far as the tenants and patrons of such buildings are concerned, the economical, operation of such elevators. Railways classify, not only their freight, but their passengers, though not, perhaps, on the lines above mentioned. The point to which it is desired to draw attention is this: In a large, centrally located building—one of the most expensive in the city—in which the speaker has had an office for the past six years, the employees of the building—scrubwomen with dirty, greasy buckets, bootblacks with large, besmeared boxes, and coal-heavers from the engine-room, with their clothes as black as soot—are allowed in every one of the ten passenger elevators of the building. This practice occasionally, in fact many times a day on some days, is a great inconvenience to the tenants and patrons of the building, as frequently they are obliged to wait for one or more trips before finding a car suitable for them to ride in to their offices. No objection is made to the employees—the men or the women—but, in rush hours, it is not pleasant to be squeezed in among the greasy buckets and blacking boxes, or against sooty jumpers, so that such practice, as with other things, hinders the regular and economical operation of the elevators.

Some months ago, the owners of the building referred to requested their tenants to make suggestions as to its better management. In reply, the speaker wrote and suggested that one elevator of the ten be set apart for employees. An order to this effect was issued, and for a few weeks carried out, but it was soon neglected, and all elevators were and are used as before. This is not on account of the want of courtesy of the elevator or hall men, but is traceable directly to the

Superintendent of the building, who, for some reason, refuses to carry out the orders of the owners. It would seem, therefore, that, in addition to those mentioned by the author, the disposition of the Superintendent is an item which should be taken into account as affecting, in the broad sense, the successful and economical operation of passenger elevators. Mr. Stanton.

WILLIAM J. BOUCHER, ASSOC. M. AM. SOC. C. E.—One of the largest and highest buildings in New York City is equipped with electric elevators, all on local service. There are ten cars, five of which run to the 26th floor and five to the 25th floor. They are of a type which was installed quite frequently ten years ago, but which has since been largely superseded. The elevators are not giving satisfaction, and the entire equipment needs rebuilding. The management, however, is confronted by the fact that they have several important tenants whom they cannot afford to lose, as they would if there were a complete shutdown. Further, the machines are located so that it is impossible for men to work among them while the others are in operation. There is seldom a day when at least one car is not out of commission—perhaps one for a part and another for the remainder of the day. Due to the overloading, which is inevitable during the rush hours, the "safety" frequently springs on the "down"; this causes inconvenience to the passengers, who have to leave the cars by means of a step-ladder, and the delay amounts to half an hour or more while inspection of the bearings and other parts is made. In the morning rush upward, the overloading is such that frequently several would-be passengers must leave the car before it can be started, and meanwhile the throngs are arriving and filling the hall, impatient to reach their offices. Mr. Boucher.

The operation of cars on "schedule," or consecutively, is no doubt the best system, its greatest feature being that the hall man knows which car to expect next, and can direct the waiting people where to stand. Something has been said of the hall man and his efficiency. A great deal depends on these men; they can give good or poor service; and be it said to their credit, they generally do all in their power to make matters go smoothly. They are often hampered by the operators on the cars, who, whether by reason of insufficient pay or pure negligence, are frequently careless, and run the cars to suit themselves by failing to stop for waiting passengers, this latter happening particularly at the top of the building.

The size or capacity of the cars has considerable to do with their efficiency, and one larger than its neighbors is a distinct drag on a system of elevators. The speaker learned to shun a car larger than its fellows, because it always started with so many passengers that their disembarkation delayed it so that it arrived at the upper floors later than the car which followed it from the ground floor.

A few years ago, a test of these elevators was conducted as a thesis

Mr. Boucher. test by students of Stevens Institute of Technology, and the following figures are taken from that test:

Offices per floor (various sizes)	39
Height, from ground to 26th floor	309 ft.
Distance between floors (generally)	12 "
Floor area of cars	36 to 44 sq. ft.
Average number of round trips per hour	16
Average number of passengers over entire trip	5.5

This last figure, however, is subject to great fluctuation, as cars frequently start with as many as fifteen passengers going up, and arrive at the top floor with one or two passengers. The tests showed:

Starting up, from ground, per hour	126 to 178 passengers.
Arriving at 26th floor	19 to 22 passengers.

Due to the peculiar system of roping these cars and counterweights, there is a great length of cable, and unless the operator is particularly skillful, there is a distinct and disagreeable "lurch" or "surge" at the stop, and many women passengers are made faint by the sensation.

Mention has been made of the newer electric installations, but it seems to the speaker as though the plunger hydraulic is giving the most satisfactory service. Of course, there is some question as to the permanency of the installation tubes placed in the ground, and wet inside and outside, and it would be very interesting if Mr. Bolton would contribute from his knowledge the latest ideas on the subject.

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AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

THE BONDING OF NEW TO OLD CONCRETE.

Discussion.*

By Messrs. H. R. Burroughs, Theodore Belzner, and James L. Davis.

Mr. Burroughs.

H. R. Burroughs, Jun. Am. Soc. C. E. (by letter).—The writer was much interested in Mr. Goodrich's paper, as it recalled a test which had come to his notice and may be worthy of mention.

The writer was required to make an inspection of a reinforced concrete building, the report having been prompted by the fact, that upon stripping the forms, the concrete was found to be porous, with many visible pockets. After considerable investigation, it was concluded that it would be necessary to remove the porous parts and refill the places thus made vacant. The expedient of bonding new concrete to old was resorted to, as the defective places had already been poured several weeks. No apprehension, however, was displayed in recommending this method of procedure, which was undertaken and, to all appearances, carried out with success and satisfaction.

In view of this undertaking and incident thereto, the Superintendent in charge conceived the idea of making a rough bonding test, more to satisfy his personal curiosity than to arrive at any conclusion. He selected a barrel and poured into it the regular concrete mixture, as used on the work, until it was one-third filled; this was permitted to set for seven days, when the second third of the capacity of the barrel was poured; this also was permitted to set for seven days, when the last third was poured, filling the barrel. The barrel was then set

*This discussion (of the paper by E. P. Goodrich, M. Am. Soc. C. E., printed in *Proceedings* for January, 1909), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

Mr. Burroughs. aside for two weeks, when the staves were stripped from the concrete, exposing the concrete mass. It was noted that the lines of cleavage were scarcely visible, the whole exhibiting an appearance of continuity and of being monolithic.

It was then attempted to break this concrete core by well-directed blows from a sledge. It required several severe and direct blows, however, to start a break, the ultimate rupture occurring, as might be expected, on a line of junction between two zones of different times of pouring.

The bond or cohesion displayed, however, seemed remarkable, inasmuch as no attempt had been made to effect a union of the parts, the surfaces not having been washed or cleaned.

Mr. Belzner. THEODORE BELZNER, JUN. AM. SOC. C. E. (by letter).—The writer has read with much interest the description of these laboratory experiments on the bonding of new to old concrete, and also the published deductions from tests made by Messrs. Perry and Mesnager.

There are many methods for bonding concrete, but the writer will confine his discussion to that in which the strength depends on the degree of adhesion of concrete to concrete.

Excellent results in bonding can be secured in the field by conscientious treatment and intelligent supervision, although such results cannot be compared with those of the laboratory, as, in the latter case, the experiments are made under conditions which are practically impossible to secure in the field.

Laboratory experiments have determined that roughening the old concrete surfaces and applying a layer of cement paste have given the best results. The writer has used this method on construction work in the New York Subway and on other engineering work. He has had occasion to examine concrete jack-arches, walls, etc., in which great care was exercised in the bonding, and while these arches appeared to be monolithic for a year or more after their construction, upon close examination, they revealed fine hair-line cracks on the surface. These cracks could be readily traced at the junction of the work of one day with that of another, indicating a break between the bond. On many other occasions he has observed breaks in the bond of concrete constructed under less careful supervision, the joints of which could easily be traced between the layers of concrete. This was proved conclusively by examining concrete which had been demolished and in which the joints had been separated. The writer also recalls several instances when engaged in locating leaks in steel-concrete construction and in jack- and roof-arches,* where surface water had percolated through the horizontal joints. These investigations have convinced him that perfect joints in concrete are a fallacy, experience having taught him that the different days' work

* *Transactions*, Am. Soc. C. E., Vol. LI, p. 130.

can always be traced, despite any precautions which may have been taken. It is his opinion that investigators should continue their laboratory experiments along this line, but before acid and other treatments are permanently adopted in the field (even though good results have been obtained on a small scale), the present methods should be continued (roughening the old concrete surfaces and applying a cement paste) until acid treatments, etc., have proved positively their practical efficiency. Mr. Belzner.

The only possible way of securing a perfect monolithic structure is by carrying on the work continuously, day and night, until finished. This can be done, at times, on small structures, but is practically impossible with structures of magnitude.

JAMES L. DAVIS, ASSOC. M. AM. SOC. C. E.—The speaker has made a small series of bonding tests for the Board of Water Supply of the City of New York, under the direction of Ernst F. Jonson, Assoc. M. Am. Soc. C. E., Engineer Inspector. Mr. Davis.

The purpose of the tests was to determine the efficiency of a proprietary bonding compound. The test specimens were beams 6 by 6 in. in cross-section, and 30 in. long. The concrete was a 1:3:5 mixture, with ordinary sand and gravel aggregate. The tests were conducted as follows:

Three beams were broken by central loading on a 26-in. span at the age of 28 days. All broke very nearly in the middle, and gave ordinary strength values. Each half of the three beams was put into a wooden mould similar to those in which they were cast, leaving one-half of the mould empty. New halves were then cast, restoring the beams to their original dimensions, making six specimens. Before casting the new halves, the fractured end surfaces of the old beams were wetted and covered with a cement grout of as heavy consistency as could be applied with a brush. In making the grout for three of the beams, the bonding compound was dissolved in the mixing water as directed by the makers. For the other three, the blanks, the grout contained none of the bonding compound. Care was taken to work the fresh concrete thoroughly in the vicinity of the joint, using a bricklayer's trowel. This expelled the air and insured contact with the old surface.

After 28 days, the beams were again broken in the same manner as before. The breaks occurred in the new concrete about $\frac{3}{4}$ in. from the joints, in one beam touching the joint at one point. There was no difference in the results where the bonding compound was used and where it was not. The result in every case was a perfect bond within the limit of the strength of 28-day concrete.

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PAPERS AND DISCUSSIONS

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THE ACTION OF FROST ON CEMENT AND CEMENT MORTAR, TOGETHER WITH OTHER EXPERIMENTS ON THESE MATERIALS.

Discussion.*

BY MESSRS. WILLIAM MAYO VENABLE, HERBERT W. HATTON, AND JOHN C. WAIT.

Mr. Venable.

WILLIAM MAYO VENABLE, M. AM. SOC. C. E. (by letter).—The writer is much interested in this valuable paper, having had experience with the effect of frost on concrete, and the effect of using salt water in mixing concrete.

In the construction of the Long Key Viaduct, on the Florida East Coast Railway, the concrete was at first mixed with fresh water only; but, owing to the expense of supplying fresh water for this purpose, so many miles from any natural source of supply, salt water was finally used, it being believed that it would not have an appreciably injurious effect. On pier work it was customary to allow the cofferdams to fill with salt water before the removal of the forms, and in less than a day after the placing of the concrete; and it is interesting to note that the tests set forth in *Experiment F* and *Experiment G* indicate that no appreciable difference in the strength of concrete mixed with fresh water and that mixed with sea water need be expected. The writer's reason, however, for preferring fresh water was doubt as to the action of the salt water on steel reinforcing rods.

The conclusion that mixing cement mortar with warm water has a

*This discussion (of the paper by Ernest R. Matthews, Esq., and James Watson, Esq., printed in *Proceedings* for January, 1909), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

permanently injurious effect, if not requiring some modification, is most important, as this is far the easiest way by which the temperature of materials for concreting in freezing weather may be raised sufficiently to cause the concrete to set. On all the sections of the sewer system now under construction in Louisville, Ky., concreting is permitted when the temperature is above 24° Fahr., but the sand, gravel and water must be separately heated before mixing. The concrete is hardly likely to be frozen after being deposited, owing to its protected location. If any person has investigated the subject of the effect of temperature, above freezing, upon the strength of concrete, and the rapidity with which the full strength is acquired, the data obtained ought to be communicated. It may be that the heating of the ingredients separately is as injurious as the separate heating of the water; or that heating the water is injurious only when it raises the temperature of the resultant mixture above a certain point. The practice of heating concrete materials, generally the water, to remove frost in gravel and sand, and thereby obtain concrete at about 60° in the forms, thus securing sufficiently rapid initial set, is quite general, and some method of heating is necessary if concreting is to be done at all when the outside temperature is below freezing. Mr. Venable.

Can any person furnish the results of a series of tests upon mortar mixed with warm water, showing to what temperature the water, or the mortar, may be raised without injury to the ultimate strength to be acquired? The writer has frequently heated water in mixing concrete to more than 80° Fahr., producing concrete at 54° or higher as it left the mixer, and has never discovered any weakness in such concrete; and if the heating of the water alone is not injurious, below a certain temperature, it is much better than heating the sand and gravel separately, because it is simpler and more economical than any other method.

HERBERT W. HATTON, JUN. AM. SOC. C. E. (by letter).—In reviewing this very excellent paper, the constructing or field engineer at once seeks to make comparisons between the respective tests recorded and actual construction under similar conditions; for after all it is the results obtained in the field under alternate conditions of frost and thaw which are of paramount importance. Mr. Hatton.

The conditions under which concrete is tested in a laboratory are vastly different from those encountered in the field, for example, suppose 100 cu. yd. of concrete to be mixed between temperatures varying from 28 to 40° Fahr.—and this is frequently the case—the main differences between concrete mixed in the laboratory and concrete mixed on the work will be:

First.—The cement used in one case has been practically kept in cold storage, and its actions in setting are retarded, whereas that used in the testing laboratory is kept practically at 60° Fahr., summer heat,

Mr. Hatton. hence crystallization begins at once upon the making of the test pieces.

Second.—The water used in mixing the concrete is usually very cold, or it is heated until it is quite hot, and, in either case, setting is greatly retarded.

Third.—The ingredients forming the concrete are either cold or they may be heated; in the one case they may be full of frost, and, in the other they may be so hot as to absorb readily large quantities of water from the cement and needed by it in crystallization.

Fourth.—Concrete mixed when the temperature is 29° Fahr., or thereabout, will get neither its initial set nor hard set as quickly as that mixed when the temperature is 60° Fahr., nor is it possible in all cases to maintain the surrounding atmosphere at a temperature of 60° Fahr. for 24 hours, except at considerable expense.

It seems to the writer that the result of the action of frost and thaw on concrete can best be obtained from actual cases in the field. It is true that practical tests, at a great deal more expense, can readily be made under actual working conditions, and possibly such tests could be made by the committee which has received the Government appropriation, set aside for the past year or more, for testing concrete beams, etc. This paper, however, is quite a valuable asset to the literature on this subject.

An actual case of recent date, where concrete was placed when the temperature varied many degrees, may be of interest. During the early part of the winter of 1908, the writer was engaged upon the construction of a large dam at Berwick, Pa., and incidental thereto, the spillway of an old dam had to be torn out and replaced. Cold weather came at a time when the entire paving, concrete apron, etc., of the spillway, had been removed, and it was necessary to repave and concrete before the spring freshets came; this necessitated the placing of the concrete covering and apron at the toe regardless of the weather conditions.

The spillway was 60 ft. wide and about 55 ft. long, with a slope of $1\frac{1}{2}$ to 1. The thickness of the concrete was 7 in. The number of square feet of concrete placed each day was too great to heat readily. During the mixing of the concrete, the temperature ranged from 28 to 40° Fahr., much of it being mixed at the lower temperature. The stone was full of frost, as was the sand, and the water came directly from the dam, ice being broken in order to get it. The cement was stored in a shed, the temperature within which was the same as the outside atmosphere.

In all cases the sand was heated by a fire, kept constantly burning within a 16-in. cast-iron pipe, the sand being spread over and around the pipe. The water was also heated until it steamed. In using the water, the practice was to empty two buckets of ice water into the large

kettle each time two buckets of water were taken therefrom for mixing concrete, and in this way the temperature of the water used in mixing was reduced so that it had no harmful effect on the concrete. Several barrels of cold water were kept close at hand for this purpose. Mr. Hatton.

The heated sand and the cold cement were first mixed thoroughly, and the stones were placed thereon, then the warm water was poured over the stones, and in this way the temperature of the water was further reduced, so that on the completion of the mixing the temperature of the mixture rarely exceeded 50° Fahr.

In no case was the mixing carried on when the temperature was below 27° Fahr., and whenever the thermometer stood below freezing the concrete was covered as fast as it was placed.

In order to ascertain how much covering of manure was necessary to insure the non-freezing of the concrete under varying conditions of temperature, the following experiment was made.

Small quantities of concrete were mixed as described, and, when placed, were immediately covered with cement bags, or with tarpaulins, upon which hot manure was spread. The temperature on two occasions dropped to 10° Fahr., but, with a covering of manure 8 in. thick, the concrete was in no case affected. Coverings 4 and 5 in. thick were found insufficient, as this small body did not contain heat enough in itself to continue to generate heat from the phosphorus, ammonia, etc., therein contained, and in two days' time the frost practically went through it.

Almost every night the temperature dropped to 24° Fahr., frequently went down to 15 or 16°, and on two occasions down to 10° Fahr., but the covering, which was 8 in. thick, on top of cement bags or tarpaulins, proved adequate, except when a rain occurred, and the covering got wet, and then even a thickness of 8 in. was insufficient, as the water dripped down through the manure, and when the temperature dropped, as it usually did after each rain, the frost went through the covering, forming a heavy white frost directly on top of the tarpaulin and also reaching the concrete. Therefore, whenever a rain occurred, in case the concrete had not already obtained its hard set, it was necessary to take off the old manure and replace it with a fresh covering.

The manure was obtained from the contractor's mules, which were kept within a few hundred yards of the operation, thus the only cost connected with the covering of the concrete was the hauling and the spreading. The farmers in the vicinity were eager to collect the manure and haul it away without cost, after it had been used for this purpose.

Nazareth cement was used on the work, and was very slow setting. In some cases, when mixed in concrete, it did not obtain its hard set in from 30 to 40 hours.

Mr. Hatton. The following method was used to determine whether, in any instance, the frost had worked beneath the covering: Before placing the tarpaulin several quarts of water were poured over the concrete where there was a slight hollow place or indentation, and this water was added to, until the concrete had taken up all it would hold. It was not necessary to add more water after the concrete had obtained its hard set, and in no case was there the slightest indication of freezing during a daily examination for five consecutive days.

In a couple of instances the writer had the manure covering removed within four days, but there was still considerable moisture in the concrete, and on the surface, due to the manure. This moisture was soon frozen, and after a subsequent thaw, the top surface could be scraped or peeled off.

The practice thereafter was to leave the covering on for at least a week or ten days. When it was removed, the concrete was in first-class condition, and apparently continued to harden uniformly. In using pick and sledge on this concrete, the writer found that, as far as he could judge, it had set just as well as if it had been mixed under the most favorable conditions.

This is directly in line with the statements of Messrs. Matthews and Watson, to the effect that if a cement or concrete obtains its hard or final set before being exposed to frost, it will, upon exposure, continue to crystallize and harden.

The hot water with the frosty stone had a tendency to create a normal condition under which the cement suffered no injury.

Considerable care should be taken, however, to protect the manure from becoming wet during rain storms. If possible, it should be covered with tarpaulins, or with planks arranged in such a manner as to exclude the water. This is a practical and yet a very inexpensive way to protect concrete in the open where a low flat surface is presented.

From this experience, the writer has no hesitation in saying that, in his opinion, if concrete is properly mixed and protected until it has obtained its final or hard set, a frosty atmosphere, with the temperature in the neighborhood of 28 or 29° Fahr., has no permanent or detrimental effect upon it.

Mr. Wait. JOHN C. WAIT, M. AM. SOC. C. E.—Some twenty-four years ago the question of the use of cement in freezing weather was discussed before this Society, and the speaker supposed it had been settled.

In South Chicago, twenty-four years ago, the temperature being from 10 to 24° above zero, the speaker put in foundations for large and heavy car shops, and then built the walls, all being laid with Portland cement mortar. In the following summer, having occasion to dig them up to put in sewer pipes, they were found to be perfectly set and strong. The speaker agrees with Mr. Stern, who has said that, if concrete

foundations can be buried or enclosed so as to let the frost come out at its leisure, the concrete will set in time. Mr. Wait.

The great power station and pulp mills at Rumford Falls were put in during extremely cold weather, but the speaker has never heard that there was any trouble with them. The foundations carry very heavy machinery, and are considered a good piece of work.

It would seem that the subject of heating is one that is much feared, and without reason. In the month of November the speaker visited Youngstown, Ohio, where he saw concrete mixed with Atlas cement and with slag cement and poured into moulds that were steam-jacketed and heated, so that the mixture boiled and steamed. In from 5 to 7 min. the concrete was taken out of the moulds and handled in the form of blocks, the walls and partitions being from $\frac{1}{2}$ to $\frac{5}{8}$ in. thick. These concrete blocks were put on cars and run into steam-rooms, where they were allowed to set. The speaker never saw any better ultimate result with concrete than these blocks, and they are now used extensively in Youngstown for the construction of houses.

Heating the water, or heating the sand or the stone does not hurt concrete materially unless it is superheated, and certainly if it is not heated above the boiling point.

It would be enlightening to hear an expression of opinion from those who may have seen this work at Youngstown, or who have had experience with heated concrete.

MEMOIRS OF DECEASED MEMBERS.

NOTE.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

JOSEPH MARSHALL GRAHAM, M. Am. Soc. C. E.*

DIED FEBRUARY 3D, 1909.

Joseph Marshall Graham was born at Crawfordsville, Indiana, in 1850. He received his education at the Kentucky State University, at Lexington. He began his railroad career, in 1873, as Assistant Engineer on the Grayville and Mattoon Railroad, now a part of the Illinois Central. From 1874 to 1875 he was Assistant Engineer on the Bedford, Springville, Owensburg and Bloomfield Railroad; and, from 1875 to 1876, he served as Chief Engineer of the same road. After that time he held the following positions:

From 1876 to 1881, Chief Engineer of the Danville, Olney and Ohio River Railroad; from October, 1881, to April, 1882, Chief Engineer of the Chicago, Texas and Mexican Central Railway; from April, 1882, to April, 1883, General Superintendent of the Danville, Olney and Ohio River Railroad; from April, 1883, to October, 1888, Superintendent of the Dakota Division, Northern Pacific Railroad; from October, 1888, to October, 1890, General Manager of the Northern Pacific and Manitoba Railway of the Northern Pacific System; from October 1st, 1890, to July, 1891, Assistant General Superintendent of the Northern Pacific Lines East of Livingston, Montana; from September, 1891, to January 1st, 1898, Superintendent of the Ohio and Midland Divisions, Baltimore and Ohio Railroad, at Newark, Ohio; from January 1st, 1898, to July, 1899, General Superintendent of the Trans-Ohio Division of the same road; from July 1st, 1899, to February, 1904, Chief Engineer of the Baltimore and Ohio Railroad at Baltimore, Maryland; from February, 1904, to 1908, Vice-President of the Erie Railroad; and, in 1908, he was made Vice-President of the New York, Susquehanna and Western Railroad, which position he held at the time of his death.

Mr. Graham was one of the pioneers in advocating and carrying out important grade-reduction work, to lessen train-mile cost; and while Superintendent of the Baltimore and Ohio Railroad, in 1893, he commenced the modification of the grades on the Lines West of the Ohio River. During the time he was Chief Engineer of that road, surveys were made, and a low-grade line was located from Baltimore to Chicago, a considerable portion of this line being constructed under his direction, viz., from Baltimore to Brunswick, Maryland; various grade-reduction improvements were made between Harpers

*Memoir prepared by Francis Lee Stuart, M. Am. Soc. C. E.

Ferry and Cumberland; a freight line was constructed around Cumberland, as well as a line from Youngstown to Akron; grade reductions were made on the Cleveland, Lorain and Wheeling Railroad, together with various grade reductions in connection with double-tracking the Lines West of the Ohio River.

When it was determined by the Directors of the Erie Railroad to relocate certain portions of its lines, in order to make it a low-grade line between Chicago and New York, Mr. Graham was called from the Baltimore and Ohio Railroad to take charge of the work as Vice-President of the Erie Railroad; and it was under his direction that plans and surveys were made for a line from New York to Cleveland with a ruling gradient of 0.2% east and 0.3% west, and the Erie and Jersey Railroad and the Genesee River Railroad, which are parts of this low-grade line, were constructed.

During 1908, Mr. Graham had relinquished some of the executive duties of his office to engage as Consulting Engineer in several large enterprises in different parts of the United States.

Mr. Graham was a man of large ideas, holding broad views of the railroad questions of his time, and his habits of mind, mature judgment, and sound conclusions on all practical topics of railway economics, made him an invaluable addition to any organization.

In 1906, the Kentucky State University conferred on him the degree of Doctor of Engineering.

In 1880 Mr. Graham married Miss Evelyn Norton, daughter of the Reverend Albert Norton, of Cleveland, Ohio, who survives him.

Mr. Graham was elected a Member of the American Society of Civil Engineers, on April 4th, 1900.

CLINTON GLENCAIRN WELLS, Assoc. M. Am. Soc. C. E.*

DIED AUGUST 16TH, 1908.

Clinton Glencairn Wells was born in Galveston, Texas, on December 13th, 1874. He attended school in Galveston, and in Lawrenceville, New Jersey. He was graduated from Princeton University, in Civil Engineering, in 1898. His first employment in his chosen profession was with the Jersey City Water Supply Company of Boonton, New Jersey, in 1899, in the capacity of Assistant to the Engineer in charge of a party making preliminary surveys and location of the works to carry out a $7 695 000 contract with Jersey City.

On May 1st, 1900, Mr. Wells accepted a position with the Electrical Commission of Baltimore City, in the capacity of Engineer in Charge

* Memoir prepared by John B. Hawley, M. Am. Soc. C. E.

of Distribution, on an underground conduit system for the reception of wires of all classes. In October, 1901, he left the Electrical Commission to accept a position with the Maryland and Pennsylvania Railway Company, as Assistant Engineer in Charge of Reconstruction Work in Western Maryland, straightening the line and laying out new curves.

On June 10th, 1902, Mr. Wells was elected City Engineer of Galveston, Texas, in which capacity he served faithfully until May, 1907, when, by reason of ill health, he was forced to resign. At the time Mr. Wells was elected the city was just beginning to recover from the great calamity of 1900, and his five years of service might be termed the period of its rehabilitation; most of the improvements, both public and private, were due to his skill and wisdom, and were made under his immediate supervision.

In December, 1904, Mr. Wells was married to Miss Josephine Kenison of Galveston, Texas, who with their only child, Clinton G. Wells, Jr., and one sister, Miss Hetty L. Wells, residing in Baltimore, Maryland, survives him.

One of Mr. Wells' friends and associates, Mr. W. A. Nicholson, in a recent letter, feelingly expresses sentiments held by all who really knew him:

"No words can be written that will fully express the purity of purpose that ever actuated this noble hearted boy. It can be truly said, that the light which filled his manly breast would permit him to 'sit in the center of night and enjoy bright day' and why Fate should deal so unkindly with him, and why he should die so young in the very spring-time of his manhood, is to me a mystery."

Mr. Wells was elected a Junior of the American Society of Civil Engineers on March 5th, 1901, and an Associate Member on March 1st, 1905.

OLIN McCLINTOCK BOYLE, Jr., Jun. Am. Soc. C. E.*

DIED AUGUST 19TH, 1908.

Olin McClintock Boyle, Jr., was born near San José, California, on July 5th, 1886. He was graduated from the grammar grades of the Willow Glen School, and entered the Santa Clara High School in September, 1898. Six months later his family removed to San Francisco, where he entered the Lowell High School. During the summer vacation of 1901 he was induced by the city editor of the *San Francisco Examiner* to become a writer on that paper. His short newspaper career was unusually promising. He rapidly rose to some-

*Memoir prepared by Charles Derleth, Jr., M. Am. Soc. C. E.

thing more than a mere reporter. Later, he accepted the sporting editorship of the *San Francisco Evening Post*, and held it until October, 1902, when he re-entered the Lowell High School, from which he was graduated in December of that year. He was admitted to the University of California in January, 1903, and was graduated from its College of Civil Engineering in December, 1906, having completed most satisfactorily the usual four-year course.

Immediately after his graduation Mr. Boyle went to the Greenwater Copper District of Inyo County, California, as Assistant to Mr. Alfred J. Cleary, remaining until the camp was closed. Returning to San Francisco, in May, 1907, he received an appointment as Instructor in the Santa Cruz Summer School of Surveying of the University of California. At the close of this session, he was employed for a short time by the Modesto Irrigation District Company, but left their employ to accept a position as Engineer with the Associated Pipe Line Company. Early in 1908 he became connected with the Union Construction Company, with headquarters in Calaveras County, California, and was in the employ of this company when he met his untimely death on August 19th, 1908.

In his short life Mr. Boyle gave promise of an unusually active and successful career. He was a man of great physical strength, and was a most industrious worker. He had a charming personality. He was constantly on the alert to be of service to others and to make others happy.

Less than two years elapsed between his graduation from his University and his death, but in that short time he gave numerous evidences of his love for his Alma Mater, and showed by his acts that he fully realized his indebtedness to the institution which trained him.

His early experiences in newspaper work had given him a taste for reading, and he was unusually well informed. In the year preceding his death, he contributed a number of short articles to the *California Journal of Technology*, wherein he showed promise as an engineering author.

It is fitting that the *Transactions* of the Society, which represents most broadly the profession he revered, should contain a statement of his life and work so that those who were associated with him may be reminded of his substantial character and loving personality.

Mr. Boyle was elected a Junior of the American Society of Civil Engineers on March 5th, 1907.

Vol. XXXV. APRIL, 1909. No. 4.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

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SOME EXTENSIVE RAILROAD SURVEYS, AND THEIR COST PER MILE.

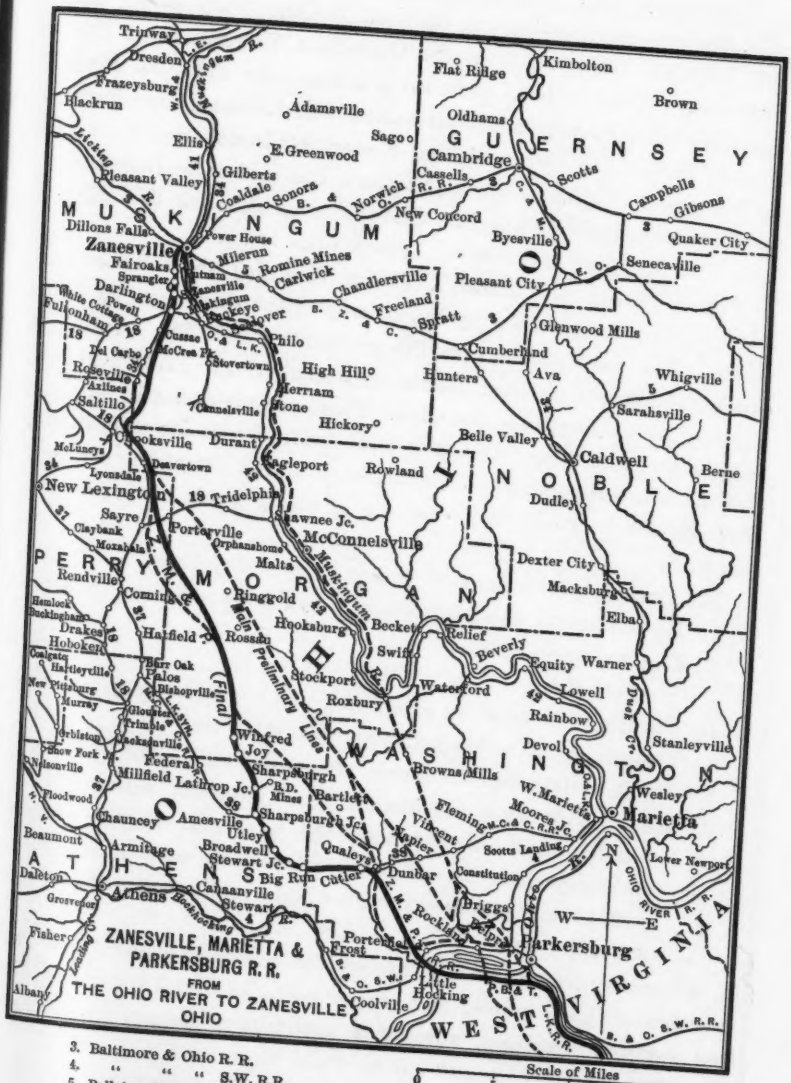
By W. S. McFetridge, M. Am. Soc. C. E.

To be Presented May 19th, 1909.

The following paper is not a theoretical discussion of railroad location. It is intended to give a general description of some extensive railroad surveys, a brief outline of methods and results, and the cost per mile. The writer does not remember having seen a similar statement covering so many miles, viz., 1 400 miles of preliminary lines and 600 miles of location; and therefore trusts the paper may prove of value. The surveys were completed some time ago, at which time the tables of mileage and cost were prepared, but not heretofore published.

Introductory.—Early in 1902 the Little Kanawha Syndicate began surveys for the extension of its lines, eastward from Palestine, W. Va., to Belington, and westward from Parkersburg, W. Va., to Zanesville, Ohio. About one and one-half years later it also took up the location and construction of a line running northward from Belington to the Pennsylvania-West Virginia State line.

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.



3. Baltimore & Ohio R. R.
4. " " " S.W. R.R.
5. Bellaire, Zanesville & Cincinnati Ry.
18. Columbus, Sandusky & Hocking Ry.
34. Cincinnati & Muskingum Valley Ry.
37. Toledo & Ohio Central Ry.
38. Marietta, Columbus & Cleveland R. R.
41. Wheeling & Lake Erie Ry.
42. Ohio & Little Kanawha R. R.

FIG. 1.

The surveys were conducted under the following charters: Zanesville, Marietta and Parkersburg Railroad, in Ohio; Parkersburg Bridge and Terminal Railroad, from the Ohio-West Virginia State line to Parkersburg (this division included a bridge over the Ohio River a few miles below Parkersburg); Little Kanawha Railroad, from Parkersburg to Burnsville; Burnsville and Eastern Railroad, from Burnsville to Belington; Buckhannon and Northern Railroad from Belington to the Pennsylvania-West Virginia State line; in all, some 328 miles of main-line location, exclusive of branch lines.

Fig. 1 (Ohio) and Plate X (West Virginia) show the country traversed and the main survey lines, many of the short lines not being shown. The lines shown give the general layout. The termini, as usual, were fixed; physical conditions also fixed the Little Kanawha River as the only outlet to the Ohio. These points decided in a general way the proposed route. Owing to local conditions, it was also believed that the heavier traffic would be west-bound, and therefore that every effort should be made to get as low a ruling grade as possible for this traffic:

The desired results may be briefly stated as follows:

1.—Easiest grades possible, especially against west-bound traffic;

2.—Lightest curvature;

3.—Shortest line;

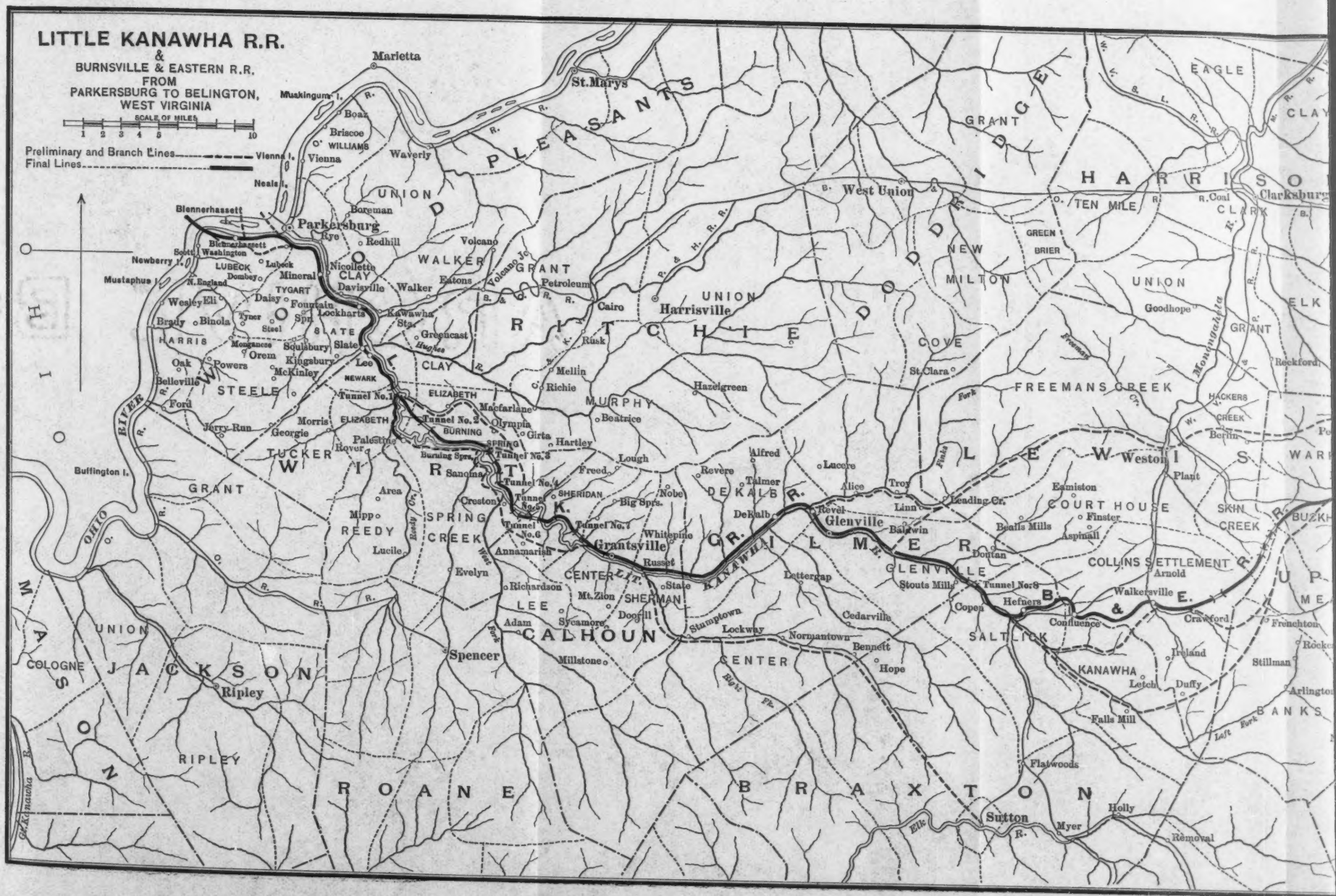
4.—Occupy to as great an extent as possible critical and strategical points, so as to block the route to other lines;

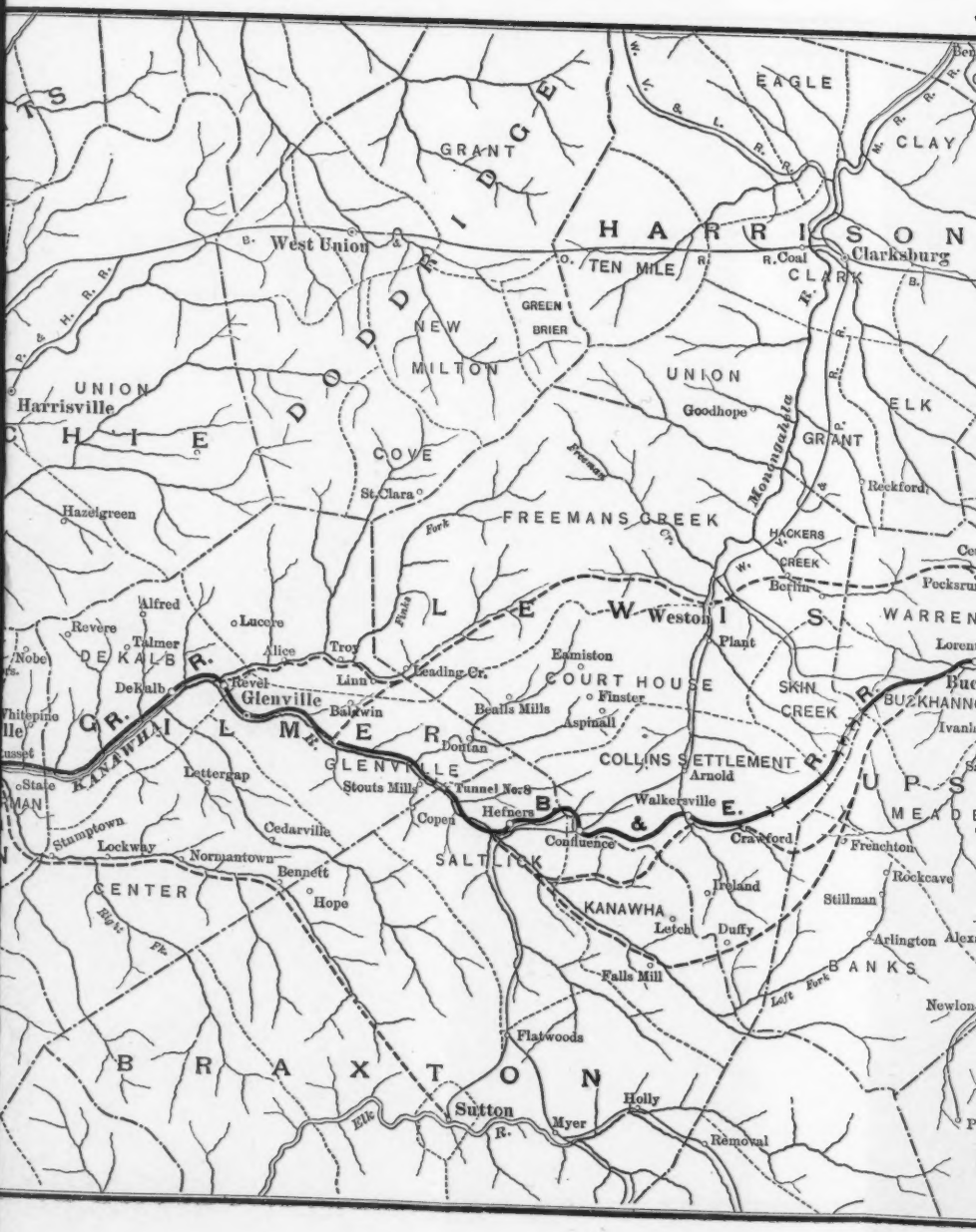
5.—Reach certain definite places previously determined on;

6.—And, taking into consideration the naturally heavy and expensive work on any line, the total cost should compare favorably with the cost of any other line, many miles longer, with heavier grades and curves, but avoiding some of the most expensive work.

All roads previously built through the adjoining regions have long stretches of 1.5% grades, and curves up to 12 and 14 degrees. The first surveys, therefore, were of a preliminary nature, in order to determine what grades and curves could be secured.

After a number of surveys, locations, and explorations had been made, it was found that the following grades and curves were possible; in Ohio, 0.5% grades, 4° maximum curve; Little Kanawha Division, 0.3% grades, 8° maximum curve; Burnsville and Eastern Division,


LITTLE KANAWHA R.R.
&
BURNSVILLE & EASTERN R.R.
FROM
PARKERSBURG TO BELINGTON,
WEST VIRGINIA
SCALE OF MILES
1 2 3 4 5 10
Preliminary and Branch Lines
Final Lines
Marietta
St.Marys
Muskingum R.
Boaz
Briscoe
WILLIAMS
Vienna
Waverly
Vienna I.
Neals I.
UNION
Boreman
Parkersburg
Rye
Redhill
Blennerhassett
Scott
Washington
Newberry I.
Mustaphus I.
LUBECK
Lubeck
Dombey
N. England
TYGART
Daisy
Wesley
Eli
Brady
Binola
Tyner
Steel
Fountain
Lockharts
Spn.
SLATE
HARRIS
Monganese
Orem
Squlsbury
Slate
Oak
Powers
Kingsbury
Lee
Belleville
McKinley
NEWARK
STEELE
Ford
Jerry Run
Morris
Georgie
ELIZABETH
Palestine
Hover
TUCKER
Buffington I.
GRANT
Area
Mipp
REEDY
Lucile
OHIO
RIVER
Nicollette
CLAY
Davisville
Walker
WALKER
Volcano
Eatons
Volcano Jc.
GRANT
Petroleum
Cairo
Kawawha Sta.
Greencast
Hughes
Rusk
Mellin
Richie
CLAY
Tunnel No.1
ELIZABETH
Tunnel No.2
BURNING SPRING
Tunnel No.3
Burning Spr.
Sanoma
Tunnel No.4
Creston
Tunnel No.5
SHERIDAN
Tunnel No.6
Tunnel No.7
Grantsville
Annamarish
Evelyn
Reedy Cr.
SPRING CREEK
West Fork
Richardson
CENTER
Russet
State
Mt. Zion
SHERMAN
LEE
Adam
Sycamore
Doolill
CALHOUN
Millstone
Spencer
Stumptown
Lockway
CENTER
Normantown
Bennett
Hope
Cedarville
Copen
Hefners
Confluence
Walkersville
Crawford
SALT LICK
Ireland
KANAWHA
Duffy
Falls Mill
Left Fork
Flatwoods
Holly
Myer
Sutton
Elk R.
Removal
Big Otter
Right Fk.
Cedar Cr.
Kanawha R.
COLOGNE
JACKSON
Ripley
RIPLEY
ROANE
BRAXTON
BANKS
Arlington
Stillman
Frenchton
Rocke
Harrisville
UNION
Goodhope
Cairo
Petroleum
Volcano
Eatons
Walker
Davisville
Nicollette
Lockharts
Kawawha Sta.
Greencast
Hughes
Rusk
Mellin
Richie
Hazelgreen
St. Clara
Beatrice
Olympia
Girta
Hartley
Lough
Freed
Sanoma
Alfred
Revere
Talmer
Lucere
Alice
Troy
Linn
Leading Cr.
Glenville
Baldwin
Revel
DEKALB
Dekalb
Nobe
Big Spr.
Whitepine
Russet
Lettergap
Stouts Mill
Tunnel No.8
Arnold
COLLINS SETTLEMENT
Walkersville
Finster
Aspinall
Bealls Mills
Duntan
GLENVILLE
COURT HOUSE
Eamiston
Plant
Weston
LEWIS
SKIN CREEK
BUZKH
Berlin
HACKERS CREEK
Reckford
ELK
GRANT
Goodhope
UNION
MILTON
NEW
GREEN
BRIER
TEN MILE
R. Coal
CLARK
Clarksburg
HARRISON
EAGLE
GRANT
DODDRIDGE
FREEMANS CREEK
Freeman
Middle Fork
West Fork
Monongahela
R. & H. R. R.
B. & O. R. R.
W. V. & L. R. R.
KANAWHA & M. R.
B. & E. R.


EAGLE
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DODDRIDGE
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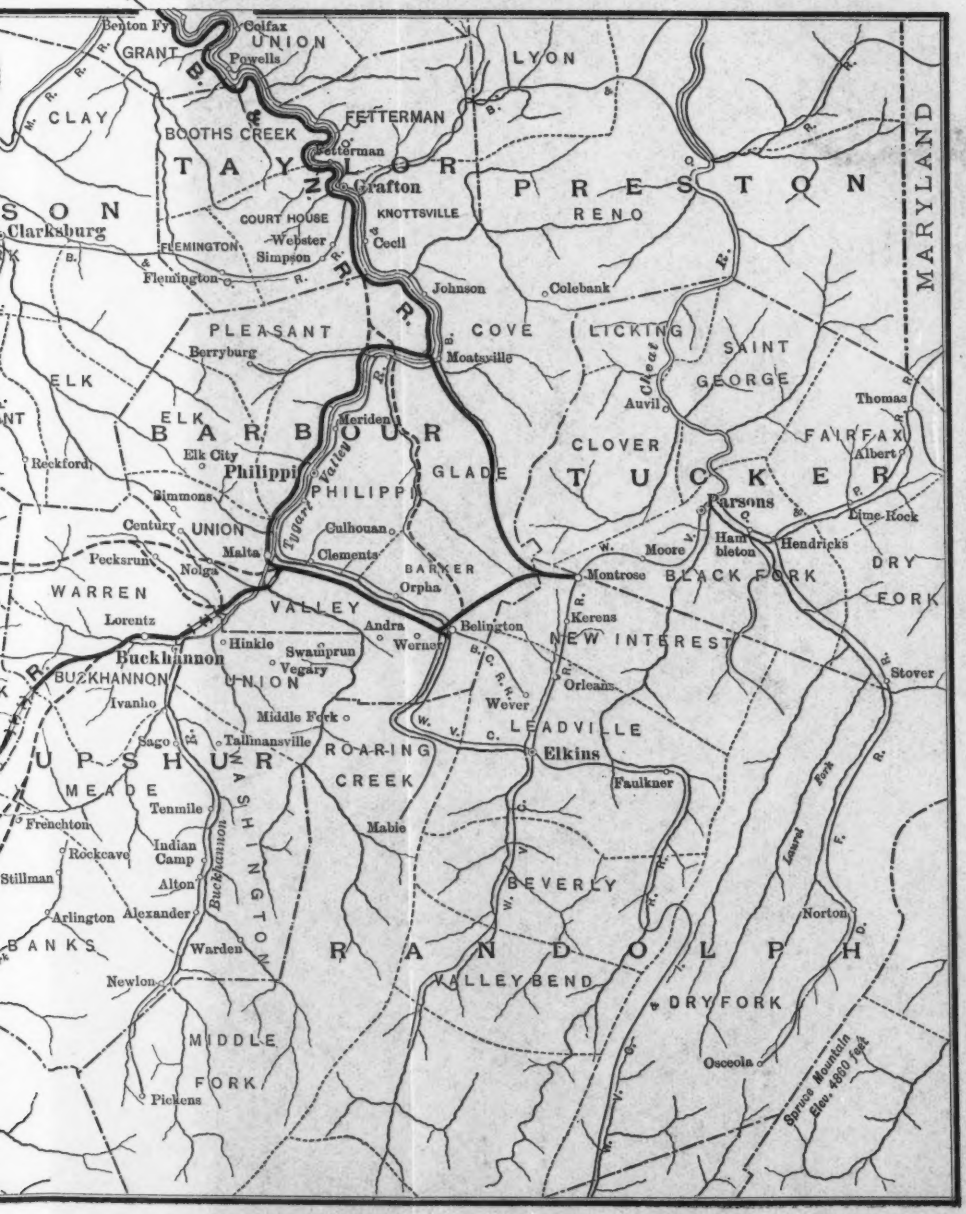
KANAWHA
Lettergap
GLENVILLE
Stouts Mills
Tunnel No. 8
COLLINS SETTLEMENT
Arnold
State
Stumptown
Lockway
Cedarville
Copen
Hefners
Walkersville
Crawford
Frenchton
Normantown
Bennett
Confluence
SALT LICK
CENTER
Hope
Ireland
Rockcave
Stillman
KANAWHA
Letch
Duffy
Arlington
BANKS
Falls Mill
Flatwoods
BRAXTON
Sutton
Holly
Myer
Elk
R.
Removal

PLATE X.
PAPERS, AM. SOC. C. E.
APRIL, 1909.
McFETRIDGE ON
RAILROAD SURVEYS.



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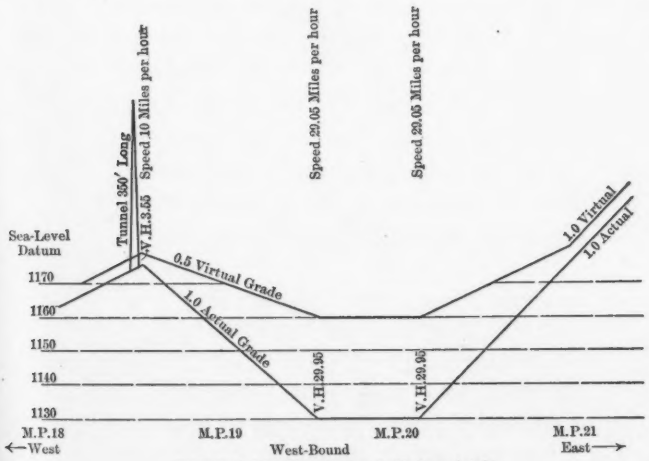
1.0% grades against east-bound and 0.5% grades, against west-bound traffic, 8° maximum curves; all grades compensated for curvature at the rate of 0.04′ per degree.

These results were kept in view in continuing the surveys, and were obtained in each case. It was desired to avoid all momentum grades, and only in one case was it found necessary to use them. This case occurred at Mile 20 on the Burnsville and Eastern Division, where such a grade was introduced in order to avoid a long détour combined with some exceptionally heavy work. At some future time this grade can be taken out if desired. In the meantime, for any ordinary reason of operation, a train need never stop there, and thus get stalled; there is no station stop, and a water-station or siding is not feasible, on account of local conditions, so that trains can always "run for the hill." Fig. 2 is a profile at this place.



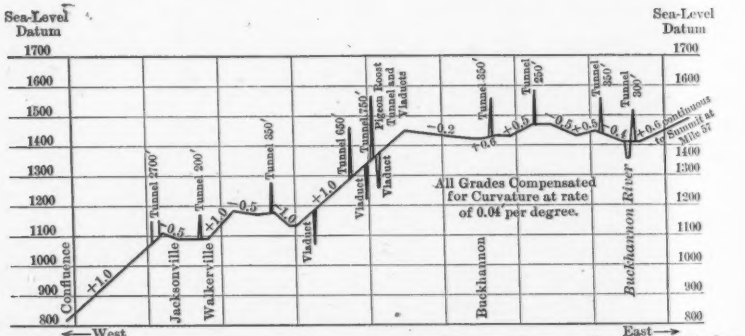
Train loaded for 0.5% grade and able to maintain uniform speed of 10 miles per hour.
BURNSVILLE & EASTERN R.R.
Profile at M.P. 20, showing Virtual Profile for West-Bound Traffic.

FIG. 2.

The grades desired were very easy for parts of the country, and required some rather long continuous grade lines, the longest being on the Burnsville and Eastern Division, where there are 1.0% grades, 7 miles and 7½ miles long, respectively, and a 0.6% grade 14 miles long, all against east-bound traffic. Fig. 3 shows a grade-line profile of this division.

To obtain these grades it was necessary to make a thorough study of the country and a detailed examination of several routes.

The topographical sheets of the United States Geological Survey were found of great value in making a broad, general study of the country. At that time the published sheets covered only a small portion of the lines, but, fortunately, this was the central part of West Virginia, where they were of particular value. It may be pertinent to state that these sheets were usually found quite accurate in regard to main summits, large rivers, and towns, but intermediate details were always more or less inaccurate.



GRADE PROFILE,
BURNSVILLE & EASTERN R.R.,
M.P. 5 TO M.P. 45.

FIG. 3.

A large number of maps of small scale (1 in. to 1 mile or even smaller) were compiled and traced from various State, county, and road maps, on which the several survey lines could be indicated. Such maps are of particular value, and should be the first ones prepared—if possible before any surveys are made.

General Layout of Surveys.—The general direction of the survey, except along the Little Kanawha Division, was almost directly across the general drainage of the country.

In Ohio a direct line between termini was first examined, but was found to be impracticable. This line marked the north and east limits of the country to be examined; certain conditions prevented the location of a line north of it. A systematic examination toward the southwest was then made, and finally a satisfactory line was developed, over which a final location was made. All the streams here lie in

deep, narrow valleys, and are exceptionally crooked. The only feasible way to traverse much of the country was to get up out of the valleys and stay out. Such a method necessitated crossing about 100 ft. above several streams, and running short tunnels between the water-sheds; it also gave the shortest line, the easiest grades, and the lightest curvature. Any attempt to avoid the high crossings or tunnels involved long détours, excessive curvature, and heavier grades. On this division more miles of preliminary lines were run, in proportion to the location, than on any other.

The main problem on the Parkersburg Bridge and Terminal Railroad was the determination of the location for a bridge over the Ohio River. The Government regulations required 90 ft. clear head-room above low water and no piers in the main channel, which necessitated a 700-ft. span. Several feasible locations were surveyed, and complete data were obtained at each place. The location finally adopted is about 5 miles below Parkersburg, and is believed to be the shortest and cheapest railroad bridge crossing the Ohio between Pittsburg and the Mississippi, the 700-ft. span practically clearing the entire channel, the eastern approach being on the only bottom lands in that region which are not subject to overflow, and the western end being against a steep bank rising rapidly from the river. The bridge and viaduct approaches are but little more than half as long as the present bridge at Parkersburg.

The Little Kanawha Division, in general, followed the Little Kanawha River. The hills rise abruptly from the river banks, the slopes often being as steep as 1½ to 1. The river is very crooked, and to follow it gave a long line with much curvature. Much distance could be saved by cutting through the country at various points, but the work was very heavy. The bottom lands are narrow and subject to overflow several times a year, the river sometimes rising 20 ft. in a day. A river line would have to be built along the steep side-hills for most of the way (this being expensive), or else be subject to inundation during high-water seasons. The hills are cut through in all directions by numerous tributary streams. The grade of the streams is from 50 to 80 ft. per mile, their length varying from 1 or 2 to 20 or 30 miles, and all of them terminate against abrupt hills. Many previous surveys of this river had been made, and practically all of them followed the river, abandoning any serious attempt to shorten

the line materially by cutting across country. The first Little Kanawha survey also followed the river, and a complete location was made. This line was run principally for information, and, shortly afterward, was abandoned. The main problem on this division was to locate such a line that its total cost would not greatly exceed the cost of a river line equally well built (though no definite amount was ever fixed for this), that its grades and curves would be as light as by river, and that it would be as short as possible. This problem required the most minute study of the land, a thorough knowledge of its character, the running of many lines, and the comparison of many estimates. On a first examination of this country one was inclined to say offhand that it was impracticable to leave the river for any material distance because the cost would be prohibitive; but when a thorough study began to show the length of line that could be saved by a mile or two of enormous work, the question had a different aspect. The only way to get sufficiently accurate data, on which to base final conclusions, was to make the actual surveys of the different routes.

One item which was always more or less an unknown quantity, but which was the deciding point between two lines, in more than one case, was the treacherous character of some of the side-hills. In a number of places (the same trouble was also encountered on other parts of the lines) the side-hills were very likely to slide. It often happened that quite a large area, and sometimes from 15 to 20 ft. deep, would move bodily down hill for several feet when disturbed by any construction work. To avoid these places as much as possible, and to estimate their probable cost when comparing lines, were difficult problems. On location, a plan sometimes used was as follows: Where the grade was near the foot of a hill, the location was thrown away from the hill, in order to avoid cutting the slopes; this gave a fill where it was not apparently necessary, but it was cheaper in the end. When the grade was high up on the hillside, the opposite was done, for the purpose of getting the roadbed on solid ground. In many cases, however, such places could not be avoided in any way, and it was often found that ground, to all appearances perfectly firm, would slide after construction started. In comparing the various lines, this tendency to slide had to be taken into account, but the estimate of cost was of necessity more or less an approximation. These slides, throughout this country, are frequently a very large item of cost in building, and are usually underestimated.

The line, as finally located, is a combination of river and cross-country line. It is 31 miles shorter than the river, in a total distance of 100 miles. There are eight tunnels, usually short, the longest being 4 000 ft. There are seven river crossings, with main spans from 100 to 300 ft.

The Burnsville and Eastern Division is in the central mountain part of the State. The highest altitude reached is 1 725 ft. above sea level. At the junction with the Little Kanawha line, the altitude is 750 ft. The lines generally run east; the mountains or hills, north and south; the drainage, generally north. There were several intermediate summits to be crossed. It was about as difficult to find the 0.5% grade west as to find the 1.0% grade east. The first line examined was up the head-waters of the Little Kanawha, up Fall Run, and through the head-waters of French Creek. The country was very rough and broken, and supporting ground for grades could not be found. This line also developed the fact that, owing to the rapid rise of the land at the eastern end, it would be necessary to use the maximum grade at once for getting out of the Little Kanawha water-shed, and then follow around the head-waters of the streams to the north, in order to get supporting ground. The line finally located was outside of the Little Kanawha water-shed.

The Buckhannon and Northern Division followed the river for about two-thirds of its length, the other third being cross-country. The general problems of location were very similar to those on the Little Kanawha and Burnsville and Eastern Divisions.

Methods Used.—Field parties were made up as follows:

	Monthly Salary.
Assistant Engineer in charge	$125 to $150
Transitman	85 " 100
Levelman	75
Rodman	65
Head chainman	50
Rear chainman	45
Rear flagman	40
Stakeman	35
Axemen (from two to five)	30
Topographer	65
Tapemen (two)	45
Draftsman (part time)	60

Camp outfits were not used. The parties boarded at houses along the line. This was often a disadvantage, on account of difficulty in getting quarters, especially for a full corps; but, on the other hand, the party could frequently make its headquarters at some town and drive to and from the work, so that probably this method served just as well as furnishing camp outfits.

Each party was given from 40 to 60 miles of line to cover, depending on local conditions.

The following abstracts from the "General Instructions" give an idea of the work required to be done by each party:

"The Assistant Engineer will receive instructions as to when and where preliminary lines will be run, grades proposed to be used, what the line is intended to develop, and the general information necessary in regard to the same. He will then be responsible for the amount and character of the work done by his party, and for the proper development of the country over which he works.

"Topography will be taken on lines as instructed."

(Topography was taken on practically all lines except in Ohio.)

"After preliminaries are run, the Assistant Engineer will be instructed over which line to locate.

"All location will be first projected on topography sheets, etc.

"No location will be assumed as final until examined and revised, if necessary, by the Assistant Chief Engineer, and also approved by the Chief Engineer."

The Assistant Engineer was not expected to spend all his time with the party, but to be with it enough to see that work was going on satisfactorily, and that lines were being run over the proper routes; the remainder of his time was spent in a thorough study of the country, picking out routes to be examined, and looking after his necessary office work and correspondence. On location and in particularly difficult country he was to be with the party almost constantly.

After the first route to be examined had been chosen, a preliminary line was run through; then the alternate routes were run, all surveys being tied together; and finally the lines required for a thorough development of all possible routes were run. Although the first preliminary line might give the grades and curves desired, and possibly be the line over which the final location was made, it was not finally determined on until the subject was fully investigated.

Sometimes all the preliminary data required over a certain route may be obtained quickly by stadia methods, but some sort of a definite

line is required from which reliable conclusions can be drawn. Especially is this true in heavily timbered, hilly country, where the range of vision is very limited, and traveling on foot is excessively wearisome, and unless such a method is used some feasible route is very likely to be overlooked. Many times on these surveys the Assistant Engineer was sent back over parts of the line to hunt out something better, quite often with success.

In locating long grades, it was preferable to start at a summit and run down hill. With a little experience, the Assistant Engineer could make a sufficiently close estimate of the amount to allow for compensation for curvature, and could run his line accordingly. In the mountainous part of the country here described this compensation amounts to about 6 ft. per mile, equal to 0.12% grade and preliminaries, for a 1.0% compensated grade, run on an 0.88% straight grade, gave the desired information.

In case of the choice of two apparently equal routes, a location was made over each, and estimates were prepared for comparison before choosing a line. In following the larger watercourses, it was usual to locate a line on either side for purposes of comparison, and in order to determine the advisability of crossing from one side to the other either to get a better line or to block the country against rivals.

Some branch lines were located, and quite an extent of line was located and afterward abandoned by reason of change of plans, but at the conclusion the miles of line actually located footed up one-third more (this is practically the correct proportion, but it varied a little more or less on different divisions) than would be required on the entire layout. This extra location was partly for comparison, but largely for reasons mentioned later. In addition to this, many miles of paper location, not located on ground, were made for the same reasons.

It may appear to some that there was much unnecessary location and running of preliminary lines, but in rough country like this, and on work of this magnitude (in 220 miles of this line there were twenty-one tunnels, the longest being 4 000 ft., five viaducts from 400 to 1 000 ft. long, and more than 100 ft. in height, besides numerous other bridges), it is time and money well spent. In no other way can the exact data be gotten, and it leaves no question as to the available routes and the grades obtainable.

The locating engineer, or others on the ground, may feel certain that a line is not feasible, but it is hard to furnish proof, both to himself and to others who are interested, but who will never be over the ground, that such is the case, except by map and profile. Again, an apparently hopeless line may show up much better than expected, and *vice versa*. In this the writer must not be misunderstood to mean that anyone can get the best location if he only runs lines enough; such is far from the case; lines can be run indefinitely without securing the desired results, unless the proper judgment and knowledge are combined with them, the location being usually the most difficult problem of railroad work. What is meant is that it is necessary to run a sufficient number of lines, preliminary and location, to arrive at correct conclusions and to get the requisite exact data to prove the conclusions.

Topography, showing contours, houses, roads, watercourses, etc., etc., was taken on practically all lines. This was taken on 12 by 18-in. sheets. The transit line was plotted each day on the requisite number of sheets; a light pencil line, at right angles to the center line, was drawn through each station, for ease in plotting the topography; elevations were marked at each station; the stations where contours crossed the center line were determined from the profile and marked; connections to other sheets were shown, and then the sheets were ready for field use. The lines were plotted to a scale of 200 ft. to 1 in. The topography was plotted in the field. A hollow drawing-board, 18 by 24 in., was used. The sheet in use was tacked to the board, and the additional sheets were carried inside. A strap around the shoulders of the topographer served to carry the board, and formed a support while plotting (Wellington's method).

This method was preferred to any other; it is quicker; saves much copying and plotting; the work can be plotted better in the field, where everything can be seen at the time of plotting; and at night the Assistant Engineer has a finished map to look over and study. The topography was taken accurately by using a metallic cloth tape for distances and a hand-level for elevations. Only in this way can one get a projected location to correspond closely with the actual one. The topography was ordinarily taken for 300 ft. on each side of the center line; at particularly difficult summits or similar places a strip from 1 000 to 2 000 ft. wide was often shown, the necessary topography being obtained by auxiliary lines. The sheets were inked in each night.

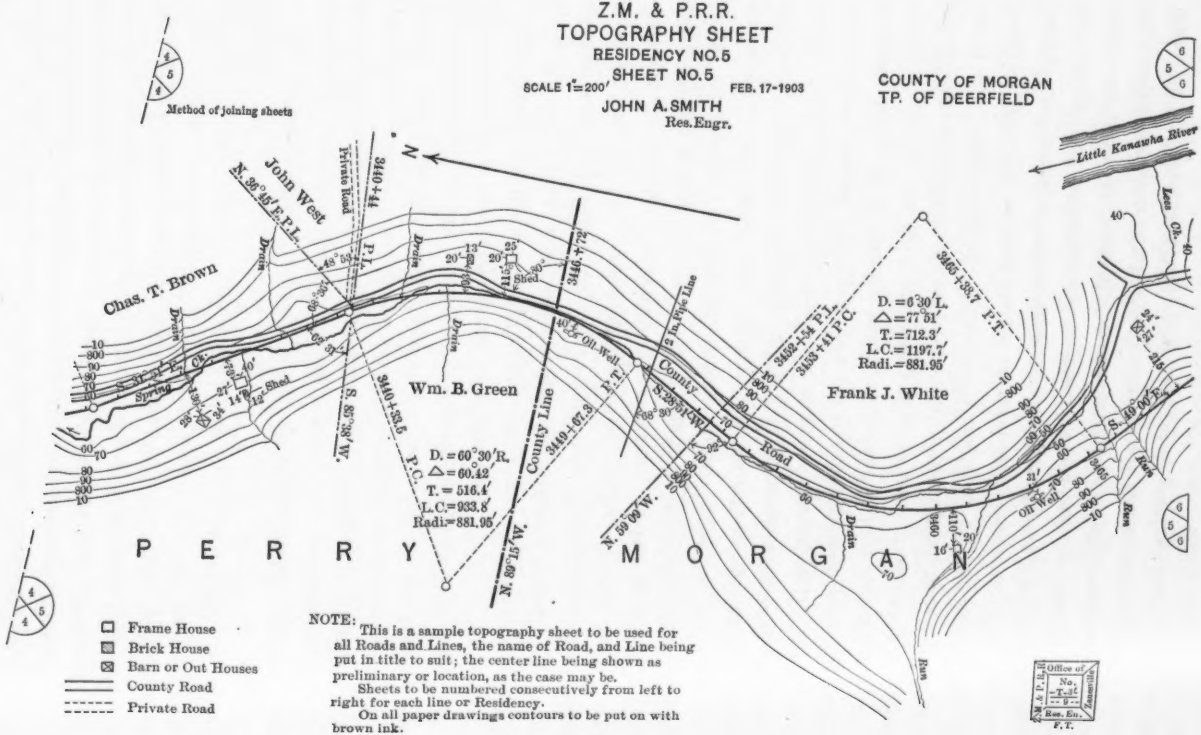


FIG. 4.

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To obtain a large general map showing all lines, the lines were carefully plotted on tracing cloth, the small sheets were fitted so as to make the center line on each sheet fit the center line on the tracing, and then the topography was traced. By this method any error in plotting or joining the small sheets was eliminated from the large map. This tracing, from which blue prints were made as required, was retained in the office, the sheets being used for rough work and field use.

The location was projected in pencil on sheets in the usual manner, care being taken to determine all angles between tangents by the calculated courses from each sheet, not by using a protractor, or by measuring intersection angles, although either might be used as a check. After the line was located on the ground, the location was accurately tied to the preliminary line and replotted on sheets; the pencil projection was then erased, if so desired.

In staking out the location, the aim was to get a profile to correspond with the projection, and not to get the lines in exactly the same relative positions shown by the projections. The lines always varied more or less in their relative positions. Also, it was often found desirable to change the location at places, giving a corresponding change in the profile; for instance, it might be decided in the field that, owing to surface conditions, certain hills could be hit harder or perhaps avoided. The most frequent change was to put the line harder into steep side-hills. When projecting lines along such places, there is nearly always a tendency to fit the line too closely to the surface and not to allow enough cutting to put the roadbed on firm ground. Owing to these facts, it is usually better to have the first location projected and run in by the field engineer, who is most familiar with actual conditions; after that the revisions and necessary changes can be taken up by the higher officers. Projections made by anyone not thoroughly familiar with the ground should be used with caution. The best location cannot be obtained without the topography and a projected line; also, it cannot be obtained on the sheets alone, without a thorough and complete knowledge of the character of the country; the two must be studied together, the field engineer at all times corresponding freely and fully with the Chief Engineer and getting his ideas and instructions.

After the first location had been made, it was studied further in

the Chief Engineer's office; if any changes were desired they were taken up with the Assistant Engineer, usually by the Assistant Chief Engineer and the Assistant Engineer going over the ground together and there studying the question. After settlement, the location was taken as final for that route, and the maps and plans were brought up to date.

B. & N. R. R.

Profile of { "A" Preliminary / Projected Location. / "L" Location / Revised Location.

from ---------- to ----------

Sta. ------ to Sta. ------

Date

---------- Chief Eng'r Name of Engineer.

B. & N. R. R.

Profile of Revised Location

on

Subdivision No. ------ Div. No. -----

Sta. ------ to Sta. ------

Date

---------- Chief Eng'r. ---------- Resident Eng'r.

B. & N. R. R.

Progress Profile.

Between sta. ------ & Sta. ---------

Div'n No. ------ Subdivn No. -------

Sect'n ------ to ------ inc.

Date

---------- Chief Eng'r. ---------- Resident Eng'r.

Note – Titles to be put in upper left hand corner of profiles.

FIG. 5.

Standards.—All curves of 3° or more had spiral approaches. These were allowed for in cross-sectioning by offsetting the slope stakes the required distance. The spiral was in standard form, so that, when the degree of curvature was known, the offset from the simple curve and the length of the spiral were obtained from tables. For simplicity and ease, all records, profiles, etc., were kept on simple curve data. When spiral curves came in tunnels, a special plan was made for each case,

showing the offsets from the tangent and the simple curve at every 10 ft. on the spiral, the alignment being kept on the tangent and the simple curve, and allowing the required offset in giving the widths for the tunnels. All grades were compensated 0.04′ per degree of curvature.

Vertical curves were inserted at all places when the change of grade was more than 0.1 ft. in 100 ft., a standard form being used in which the length of the vertical curve varied directly with the change of grade.

The usual weekly reports, maps, and profiles were sent in by the Assistant Engineers. Standard forms were used for all notes, maps, profiles, plans, and reports. The standard rules were first issued as typewritten copies, and standard forms were adopted from time to time as needed; later, they were compiled and issued in book form, being blue-printed from tracings, making 22 pages of general rules and information, together with 24 standard forms for notes and plans. It was found to save much work and time in the Chief Engineer's office to have all data in uniform shape; much correspondence was likewise avoided, as employees could readily find sizes, scales, etc., for plans, and also other information. Therefore they were not compelled to write to headquarters, and thus avoided the delay of two or three days required for letters to go and come.

A few of the standards used extensively on location are here shown.

Fig. 4 shows the standard topography sheet, the same form being used on preliminary and construction lines. After the line was finally determined, sheets were prepared showing that line only, and the title was made to suit the construction work shown.

The standard titles for profiles are shown by Fig. 5, and the standard titles for field books, by Fig. 6. The standard forms for transit and level notes are shown by Figs. 7 and 8, respectively. Fig. 9 shows the standard form for a location profile, and Fig. 10 shows the standard form for a situation survey for a bridge. When first prepared the structure was shown in pencil, and was not inked in until the detailed design had been made by the Consulting Bridge Engineers, A. P. Boller and H. W. Hodge, Members, Am. Soc. C. E. The standard form of report of openings required is shown by Fig. 11.

Cost.—The greatest number of miles of preliminary line run in one day by one party was 7, and of location, 4½. The location averaged slightly more than 1 mile per day per party, except on the Burnsville and Eastern and on the Buckhannon and Northern lines, where it

L. K. R. R.

Transit or Level } Notes on Prel. line "A" from Grafton to Moatsville, Sta. 0 to 750.

Date_____ Asst Eng'r or Levelman.

Book No._____

L. K. R. R.

Transit or Level } notes on "L" Location via "A" Prel. from Grafton to Moatsville, Sta. 0 to Sta. 748

Date_____ Asst Eng'r or Levelman.

Book No._____

L. K. R. R.

Transit or Level } notes on revised location on Subdiv'n No.____ Div'n No.____ Sta._____ to Sta._____

Date_____

Book No._____ Resident Eng'r

FIG. 6.

TRANSIT

Location Name or Letter of Line from (Town) to (Town) Sta. to Sta.

Stations.	Deg Curve	Deflection.	Delta.	Calculated Bearing.	Magnetic Bearing.
401	Tang 7465'				
+50					
400				S79°08E	S78°45E.
+53 P.T.		7° 06'			
399		6° 26'			
+50		5°48½'			
398		5° 11'			
+50		4° 33½'			
397		3° 56'			
+50		3° 18½'			
396		2° 41'			
+50		2° 03½'			
395		1° 26			
+50		0° 48½'			
394		0° 11'			
+85 P.C.C.	2° 30' L.	3° 00'	14° 12'		
+50		2° 39'			
393		2° 09'			
+50		1° 39'			
392		1° 09'			
+50		0° 39'			
391		0° 09'			
+85 P.C.	2° L.		6° 00'		
+50					
390				S 58° 56' E	S 58° 35' E.

Field books to be paged and indexed.

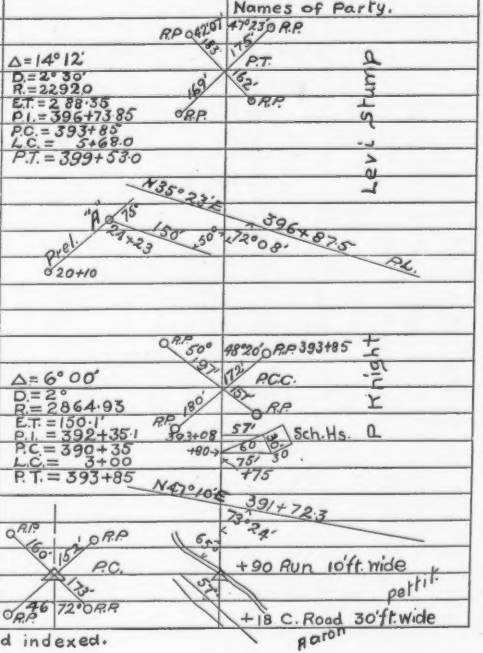


FIG. 7.

Level

Location Name or Letter of Line from (Town) to (Town) Sta. to Sta.

Stations.	Back Sight.	Height Inst.	Foresight.	Elevation.	Grade.	Cut or Fill.	
B.M.				100.00		B. M. N 2 10. on OAK. 20 ft. Left s/a. 910+50	Date.
Inst.	10.04	110.04					Names of Party.
910			5.2	104.8	97.00	c. 7.8	+35 H.W.= L.W.=
+40 Run			12.4	97.6	97.20	c. 0.4	+45 Run 10 ft. wide.
+90 Road			3.3	106.7	½ 97.45	c. 9.2	44/ Road 30 ft. 11+05
911			2.5	107.5	97.5	c. 10.0	
T.P.			11.94	98.10	+ 5/10		
Inst	0.72	98.82					
912			0.8	98.0	98.0	Grade.	
+50 P.C. S.L.			1.2	97.6	98.15	F. 0.6	
913			5.3	93.5	98.30	F. 4.8	
+50			8.5	90.3	½ 98.45	F. 8.2	
914			4.6	94.2	3/10 98.60	F. 4.4	
+50 P.T.			3.6	95.2	98.75	F. 3.6	
915							

FIG. 8.

TRANSIT

Location Name or Letter of Line from (Town) to (Town) Sta. to Sta.

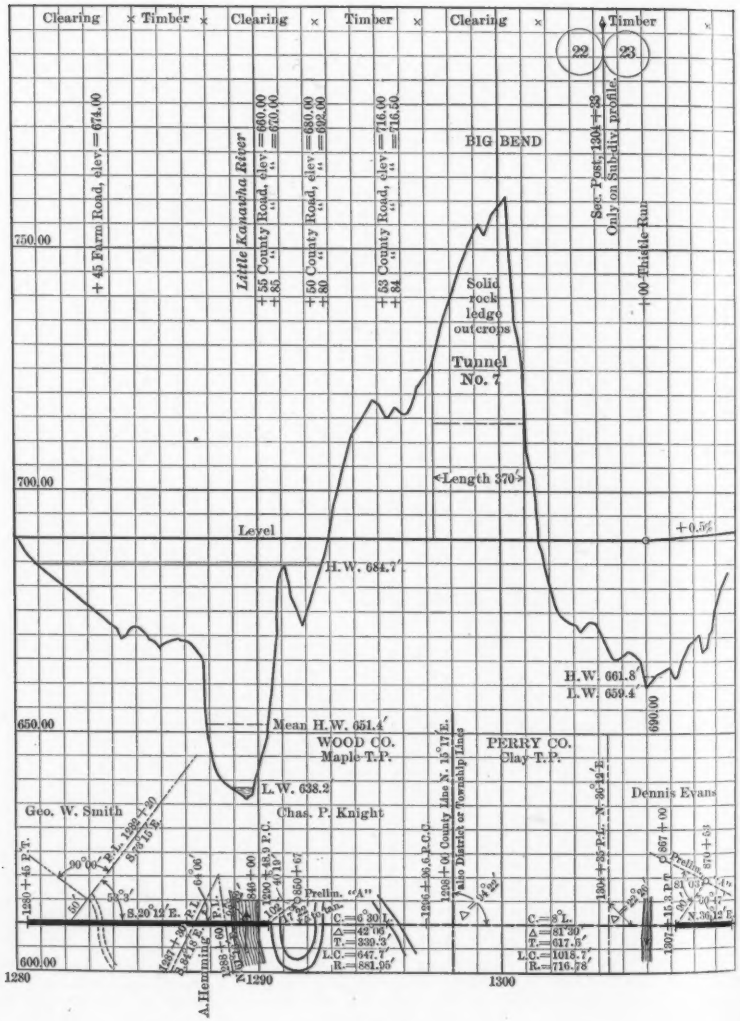
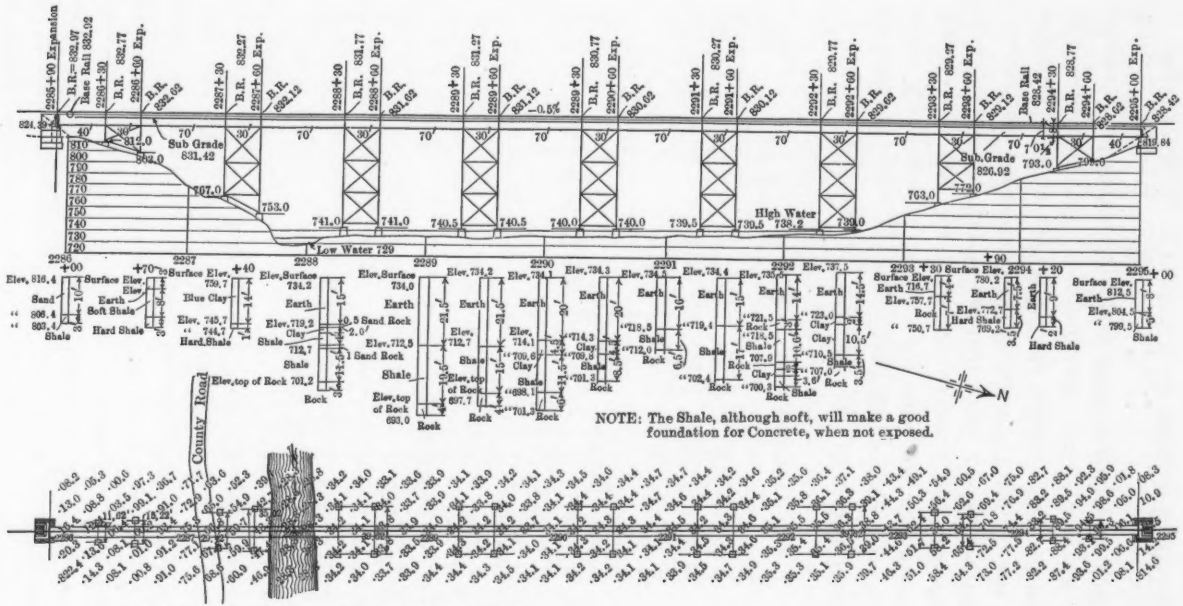

Clearing
Timber
Clearing
Timber
Clearing
Timber
22
23
+45 Farm Road, elev. = 674.00
Little Kanawha River
+55 County Road, elev. = 660.00
+85 " " 670.00
+50 County Road, elev. = 680.00
+80 " " 692.00
+53 County Road, elev. = 716.00
+84 " " 716.50
Sec. Post, 1304 + 33
Only on Subdiv. profile
+00 Thistle Run
BIG BEND
Solid rock ledge outcrops
Tunnel No. 7
Length 370'
750.00
700.00
Level
+0.5%
H.W. 684.7'
H.W. 661.8
L.W. 659.4
690.00
650.00
Mean H.W. 651.4'
WOOD CO.
Maple T.P.
PERRY CO.
Clay T.P.
L.W. 638.2
Geo. W. Smith
Chas. P. Knight
Dennis Evans
1280 + 45 P.T.
P.I. 1288 + 20
S. 20° 12' E.
1296 + 36.6 P.C.C.
1296 + 01 County Line N. 15° 17' E.
also District or Township Lines
1304 + 33 P.I. N. 36° 12' E.
1307 + 15.3 P.T.
Prelim. "A"
C. = 6° 20' L.
Δ = 42° 05'
T. = 339.3'
L.C. = 647.7'
R. = 881.95'
C. = 5° L.
Δ = 81° 30'
T. = 617.5'
L.C. = 1018.7'
R. = 716.78'
N. 36° 12' E.
600.00
1280
1290
1300

FIG. 9.



ZANESVILLE MARIETTA AND PARKERSBURG R.R.

PLAN AND PROFILE OF

PROPOSED CROSSING OF SUNDAY CREEK

SUB DIV. 3 STATION 2287+ 92

SCALE OF FEET

0 20 40 60 80 100

Chief Eng'r

FIG. 10.

averaged ¾ mile. Stakes were set every 100 ft. on tangents, and every 50 ft. on curves. Special pains were taken with the instrument work and measurements, in order to avoid the chance of serious errors in the center line after construction commenced. The speed of location parties was usually limited by the amount of clearing that could be done, but the number of curves and the rough character of the ground were also large factors in limiting the speed.

Each party cost from $35 to $40 per day, being allowed all expenses in addition to salaries.

Table 1 gives the cost per mile of the completed surveys. It is to be noted that this is the total cost, and includes office rent, purchase of instruments and supplies, general expenses, all salaries, field expenses, and the preparation of final maps, plans, profiles, and estimates, with everything in readiness to make contracts for the line.

TABLE 1.

Company.	Amount spent.	Miles of Surveys:			Average cost per mile.	Average cost per mile of location.
		Preliminary.	Location.	Total.		
(1)	(2)	(3)	(4)	(5)	(6)	(7)
L. K. R. R.	$25 076.83	428.19	193.85	622.04	$40.31	$129.36
Z. M. & P.	19 812.77	509.03	105.23	614.26	32.25	188.28
B. & E. R. R.	20 466.68	241.75	113.70	355.45	57.58	180.00
P. B. & T. R. R.	6 651.98	84.56	38.17	122.73	54.20	174.28
B. & N. R. R.	19 249.94	162.51	151.29	313.80	61.34	127.23
Totals	$91 258.20	1 426.04	602.24	2 028.28	$45.00	$151.53

Column 7 gives the cost per mile of actual location, including preliminary lines. Columns 3 and 4 show that there were from 2 to 5 miles of preliminary lines run for each mile of location, except on the Buckhannon and Northern line. Table 1 also includes 302 miles of check levels, the cost being distributed among the various accounts. The data for the Parkersburg Bridge and Terminal line include surveys and soundings for the Ohio River Bridge. The cost per mile includes the topography on practically all lines, except on the Zanesville, Marietta and Parkersburg line, where it was taken only on the located lines.

The cost shown in Table 1, being the total charge against engineering from the inception of the project to the beginning of construction,

STANDARD FORM OF REPORT OF OPENINGS REQUIRED.

Z. M. and P. R. R.

List of Proposed Openings on Sub-Division No. 2. Sections 1 to 10 inclusive.

Sta. 1610 to 2138

Station.	Name of Opening.	Acres Drained.	Elev. of High Water	Elev. of Low Water.	Area Sq. Ft. High Water.	Proposed Opening.	Adopted Opening.	Remarks.
1611+75	Sunday Creek	Not Estimated; probably 60 Sq. Mls.	738.2	729.0		Viaduct.		See Situation Survey.
1614+15		5		730.0				Ditch to 1611+75
1815+00	Federal Creek	900	740.0	734.0	125	12' Arch	Definitely Determined by Higher Officer; Other Cost or Resident Engr.	Co. Road has 14' Arch Drainage Area Cleared Stream has rapid fall. Timbered
1992+60	Bill Run	300	742.0	739.0	45	D-4X6		Stream Discharges Much Driftwood.
2001+15	Dry Run	120	743.0	742.0	20	6X8		

Note.

No absolute formula used to determine size of opening mostly determined by size of existing openings and Judgment of Engineer.

Following Formula used as basis (Myer's)

Area of Opening in Sq. Ft. = $C\sqrt{\text{Drainage Area, in Acres.}}$

Max. for C = 4.0± Ordinary Min. - C = 1.0

Min. so as not to give less than 1 sq. ft. per 12 Acres; for the small areas.

Where bridges are required, the opening being determined by economical design of structure.

John Jones
Resident Engr
Fox, Ohio.
May 24, 1903.

FIG. 11.

contains a few items which might well be charged to other accounts than location. Instruments purchased could be a credit; some elaborate property surveys and bridge surveys could be charged to construction, but they probably are not large enough to have much effect on the cost per mile. If taken into account, they would reduce the cost. The cost on the Little Kanawha and on the Burnsville and Eastern Division was increased considerably owing to much work being done during a bad winter, when the weather was very unfavorable. The cost on the Parkersburg Bridge and Terminal line was increased by a large amount of property surveying in the city, and by the surveys for the bridge.

The cost on all the West Virginia lines was increased by the immense amount of chopping and clearing necessary. When mountain laurel was encountered, all the axemen that could be worked could not keep a location party moving.

The influence of the weather is a large item in the cost of these surveys; a line run in the middle of winter may easily cost one-quarter more than if run during more favorable weather.

The average cost of one mile of preliminary or location survey, determined from a detailed study of the daily reports of field parties, of office work done, and similar data, is shown by Table 2 (to the nearest dollar), and is believed to be very close to the actual figures.

TABLE 2.

Company.	Average Cost of One Mile:		
	Of preliminary.	Of location.	Of location, including preliminary.
L. K. R. R.	$25	$74	$99
Z. M. & P. R. R.	23	79	102
B. & E. R. R.	35	105	140
B. & N. R. R.	31	94	125

The figures in Table 2 include all expenses, as in Table 1.

Table 1 shows a large variation in the cost of surveys on different divisions, the cost varying from $128 to $188 per mile, with an average of $151. On the assumption that lines located for comparison or similar purposes should be included in the average, one-third should be added to these amounts, as previously noted; the cost would then be as follows:

Low	$171 per mile.
High	251 " "
Average	202 " "

Throwing out of the account the mileage of abandoned lines, branch lines, etc., and charging the entire cost to the main line, terminus to terminus, would give $\frac{\$91\,258.20}{328} = \$278.23$ per mile, which would be rather expensive. This, however, is not a fair assumption, and should not be considered, because many miles of lines not needed to determine the main line were located for other reasons and purposes. Therefore, the plan of throwing out only duplications, for comparisons, as shown in the preceding paragraph, gives the correct average cost per mile for the development of the country, including actual comparative locations where needed. It should also be borne in mind that a large proportion of this duplication was necessary, owing to the laws of West Virginia, which require an actual line, located on the ground, and a complete map and profile of that line to be filed with the Secretary of State, and at the county seat, before a railroad company has any rights, of priority or otherwise, to that route or line. This required complete locations for all proposed branch lines, a large number of which were located, and also a complete location over any route for which it was desired to obtain rights. For these reasons, the lines located account for the excess of the mileage over the actual length of the main line.

On the basis of Table 2, it may be assumed that, where the route has been previously determined within such narrow limits that the preliminary and location lines are of equal length, the surveys will cost from $100 to $140 per mile. This is borne out by the results on the Buckhannon and Northern line where the location and preliminary lines were practically equal and the cost was $127 per mile.

These two statements may be combined and put in the following form:

To locate one mile, including an equal length of preliminary lines, cost from $100 to $140; average	$115
To locate one mile, final location, including from two to five times as great a length of preliminary lines, cost from $128 to $188; average	151

To locate one mile, final location, including from two to five times as great a length of preliminary lines, and one-third of a mile of location for comparison, cost from $171 to $251; average..... $202

A tabulation of the mileage of the Buckhannon and Northern line, with reference to the actual length of line to be built, and showing how the results agree with the averages deduced from Table 1, is as follows, the Buckhannon and Northern line being used because the conditions there make it the best average of "all conditions" encountered on the various lines.

Total miles located.....		151.29
Miles of main line contracted for.....	80	
Miles of main line not contracted for.....	4	
Miles of connecting line located, but which may or may not be built, about.....	26	110.00
Making actual miles.....	110	
Leaving duplications, comparisons, etc.....		41.29 miles.

110 miles cost $19 249.94 = $175 per mile.

Results.—The results obtained by studying and surveying the neighboring country, will amply justify the initial cost, and, without doubt, the final results will prove that it is cheaper in the end. A few thousand dollars judiciously spent on surveys will be more than made up for in construction, not only in the actual cost, but in a better line.

That the desired results seem to have been obtained, may be shown briefly as follows:

1.—Easiest grades: 0.5% west, 1.0% east (all the latter in the 25 miles near the eastern end), other roads using 1.5 per cent.

2.—Curvature: 8° maximum, as against 12° on other roads.

3.—Short line: This is seen best on the map. The Little Kanawha line is 31 miles shorter than the river, the only water route. The Ohio line is 16 miles shorter than the existing lines between termini.

4.—Occupation of critical points: Another line, since built, through the Burnsville and Eastern Division (the Burnsville and

Eastern line never having been constructed) and a few miles on Little Kanawha Division, had to locate on and occupy said lines for about 15 miles, although using 1.5% grades and 12° curves. It could not have been built for its present cost without making arrangements with the Burnsville and Eastern and Little Kanawha Railroads.

5.—The definite places previously determined on were, of course, reached.

6.—It being decided, after many surveys—by which the advantage of this line proved so apparent—to build the line as described, no complete estimate of the cheapest possible line in first cost was prepared, but a sufficient number of estimates were made to show that the line as located would certainly fulfill this requirement.

All the work was carried out under S. D. Brady, M. Am. Soc. C. E., then, as now, Chief Engineer of all these companies, who gave much personal attention to the work, and to whom the writer is indebted for assistance and information in preparing this paper, and for permission to publish the maps and costs.

The writer was Assistant Engineer on the Little Kanawha Division, in charge of location; on its completion he became Assistant Chief Engineer of all the companies, both on location and construction.

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AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

TESTS OF BUILT-UP STEEL AND WROUGHT-IRON COMPRESSION PIECES.

By Arthur N. Talbot, M. Am. Soc. C. E., and Herbert F. Moore, Esq.

To be Presented June 2d, 1909.

The principal tests described in this paper were made on the following compression pieces: (*a*) a steel column (called Column No. 1) built up of angles, plates, and lattice bars, all the parts being light; (*b*) four wrought-iron bridge posts which had seen long service in a bridge truss; and (*c*) three posts and a top chord in a railroad bridge under service. The tests of (*a*) and (*b*) were made in a testing machine; for (*c*) a locomotive and cars formed the load. Auxiliary tests were made on lattice bars and other parts.

These tests were taken up with a view of determining experimentally: (1) something of the variation of stress, both longitudinal and lateral, throughout the channel members of the compression piece; (2) something of the amount and distribution of stress in the lattice bars of columns, and the action of similar bars under separate test; and (3) the general relation between the component parts and the column as a whole. The slenderness ratio of the columns, $\frac{l}{r}$, ranged from 38 to 66. It will be seen that instead of following the more

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

common effort to determine the effect of length of column, emphasis was placed on measuring the distribution and range of stress over the various parts of the column.

It is unnecessary to state that built-up compression pieces (whether long or short) are not perfect, the natural imperfections of the component parts being increased in the process of fabrication. To non-homogeneity of structure and lack of straightness in the component angle or channel are added such further imperfections as kinks and eccentric connection of parts, which go to increase the opportunities for local flexure in the component parts and for flexural stresses in the column as a whole. An attempt has been made in these tests to measure the deformations in the presence of such conditions, and to find the general distribution of stress. In view of the many limitations surrounding such tests, the results are to be taken as suggestive and qualitative, and not as exact determinations.

Results of Tests.

The following observations on the results are given here with a view of showing the trend of the tests:

I. Variation of Stress Throughout the Length and the Cross-Section of the Channels and of the Column as a Whole.—Channel members show evidence of considerable local flexural action, such as may be produced by lack of straightness or by any method of applying the load eccentrically. This is especially true in the flimsier column.

The condition of flexure varies markedly throughout the length of the member, in some cases the maximum compression in one cross-section being at the extreme fiber on one side of the channel, and in a near-by section the other side of the channel showing the excess of stress.

There were indications of sudden changes in the relative amount of stress carried by the two channels at near-by sections, indicating general flexure of the column.

The measurements made indicate in a number of cases stresses in the extreme fiber from 40 to 50% in excess of the average stress, and in some cases even higher.

The amount of eccentricity necessary to account for the increase of stress found in individual channels, based on lack of straightness and the ordinary theory of flexure, is relatively small.

II. Stress in Lattice Bars.—The amount of deformation observed in lattice bars is relatively small.

The measurements indicate a stress in the lattice bars which would be produced by a transverse shear equal in amount to 1 to 3% of the applied compression load, or to that produced by a concentrated transverse load at the middle of the column length equal to 2 to 6% of the compression load.

III. Tests of Lattice Bars.—Tests of lattice bars for load-carrying capacity show that the usual form of bar is a very inefficient compression member when loaded eccentrically through a riveted connection.

Under common conditions of loading, the maximum fiber stress may be as much as three times the average stress.

In the compression tests of single lattice bars, the ultimate strength was in no case as much as one-half of the elastic limit of the material.

IV. Relation between Component Parts and the Column as a Whole.—It is a question whether the component members of a built-up column act together to form an integral compression piece, especially in resisting oblique distortion.

The distribution of stress under working loads, and even up to incipient failure, may be quite different from that which exists when the column becomes crippled. This is due to the yielding of the more strained parts after the yield point is reached at any fiber, and the consequent redistribution of stress.

No relation has been found between the stresses actually observed and the stresses computed by column formulas.

The sudden failure of a test column at a relatively low load by buckling of lattice bars is accounted for when the amount of transverse shear developed in other test columns and the strength of lacing found in lattice-bar tests are taken into consideration.

Laboratory Tests of Columns.

Description of Columns.—One steel column and four wrought-iron columns were tested. The steel column was built especially for the purpose of this test by the American Bridge Company at the Lassig plant. The wrought-iron columns were halves of bridge posts taken from an old bridge of the Chicago, Burlington, and Quincy Railroad.

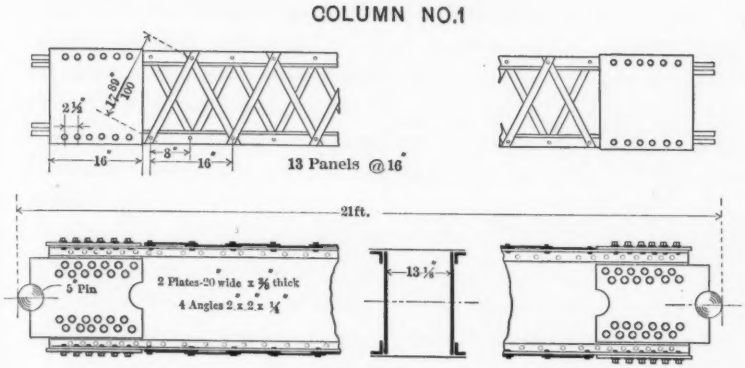

FIG. 1.

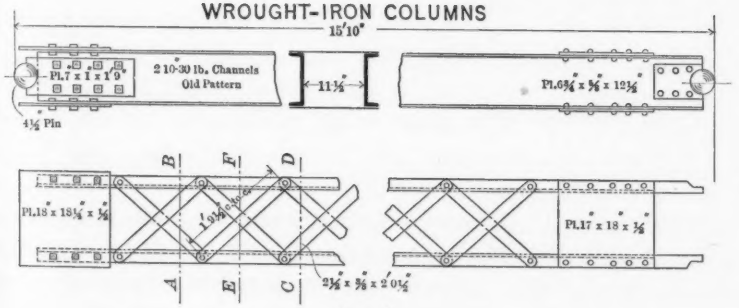

FIG. 2.

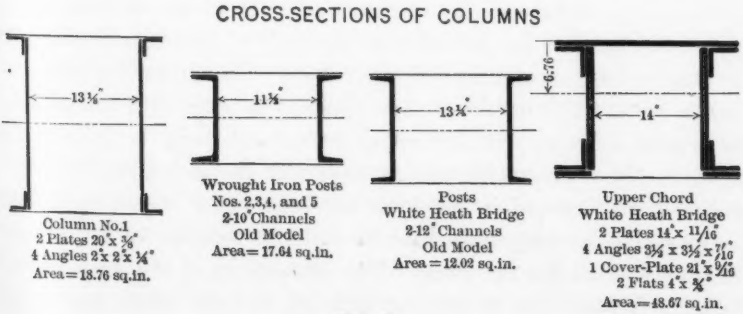

FIG. 3.

TABLE 1.—Data of Columns.

Column designation.	Column No. 1.	Wrought-Iron Posts.		White Heath Bridge.	
		Nos. 2, 3, 4, and 5.	Retests 2a and 4a.	Posts $U_3 L_3$ N., $U_3 L_3$ S.	Upper chord $U_3 U_4$ south.
Area of column section, in inches	18.76	17.64	17.64	12.02	48.67
Length, center to center	21 ft.	15 ft. 10 in.	14 ft. 7 in.	25 ft.	19 ft. 10 in.
End conditions	Pin parallel to lacing.	Pin parallel to lacing.	Pin parallel to lacing.	Lower end pin, upper end riveted.	Riveted.
$\frac{l}{r}$, axis parallel to lacing	37.8	43.5	40.1	66.1	40.7
$\frac{l}{r}$, axis perpendicular to lacing.	37.2	41.2	38.0	41.0	29.6
$\frac{l}{r}$ of each flange member, axis perpendicular to lacing for full length of column	593	400	367	416	.....
$\frac{l}{r}$ of each flange member, axis as before, for distance between adjacent lacing rivets.	37.7	33.7	33.7	22.2	.....
Lattice bars, dimensions of section, in inches	1 by $\frac{1}{4}$ in. and $1\frac{1}{2}$ by $\frac{7}{16}$ in.	$2\frac{1}{2}$ by $\frac{3}{8}$ in.	$2\frac{1}{2}$ by $\frac{3}{8}$ in.	$2\frac{1}{4}$ by $\frac{3}{8}$ in.	$2\frac{3}{4}$ by $\frac{3}{8}$ in., on bottom.
Kind of lacing	Single.	Double.	Double.	Double, riveted at crossing.	One cover-plate.
Angle of lattice bar with axis of column	63° 30′	45°	45°	45°	45° on bottom.

Table 1 gives the dimensions of the columns. The details are shown in Figs. 1 and 2; and Fig. 3 shows the cross-sections. The steel column (Column No. 1) was designed with relatively thin component parts and relatively large outside cross-section dimensions. It will be seen that this column is not so stocky as the usual bridge column. This section was chosen because it seemed to offer better opportunities under the conditions of the test for the study of the distribution of stress, longitudinal and lateral, and especially of the stresses developed in the latticing, than a less flimsy column. In the earlier tests, the lattice bars of Column No. 1 were fastened in place with turned bolts in reamed holes so that one set of bars could easily be replaced by another. After several tests were made with the lacing thus fastened, the bars were riveted in place for the later tests. The turned bolts in reamed holes apparently held the column together in the column tests as well as did the rivets. The proportions of the wrought-iron bridge columns were in no way unusual, and the posts represented good practice at the date of construction of the bridge. The columns

became available through the replacing of the bridge with a heavier structure; they were apparently in good condition.

All the columns were pin-ended, the pin being parallel to the plane of the lacing. The details of the ends are shown in Figs. 1 and 2. The wrought-iron posts were cut in two. One end was left as used in the bridge, and bearing plates and batten plates were bolted to the other end.

Procedure of Tests.—The method used in studying the distribution of stress was to measure at various parts of the column, by means of extensometers, the deformation over short distances, and its variation over a section perpendicular to the axis of the column. The extensometers were placed outside of the flanges of one of the channels in such a way that the shortening at any point of the cross-section of the member could be determined on the hypothesis that in this part of the channel a plane section before loading remained a plane section after the load was applied. It is seen that this hypothesis is not dependent upon the integrity of the column as a whole. A similar arrangement was made in observing deformations in lattice bars. The position of the instruments is shown by Fig. 1, Plate XI. Two sets of instruments were used, and sometimes more. Generally, they were placed on the front and back flanges of one channel and then on the other, but sometimes on the front or back flanges of two channels at the same time. This procedure necessitated the removal of the instruments to new positions after several applications of the load, and, in the complete test of any column, involved several hundred applications of the load.

In all the tests to determine the distribution of deformations, care was taken not to exceed the elastic limit at any point. Under these conditions, it is considered, for the purposes of the discussion, that the stress developed at any point is proportional to the deformation. The ratio of the intensity of the stress to the deformation per unit of length, therefore, will be taken to be equal to the modulus of elasticity of the material.

Extensometers.—In the observations of deformation in the channel members of the columns, the extensometers used in most of the tests were Ames test gauges mounted on suitable frames. These gauges were graduated to $\frac{1}{1000}$ in., and were read by estimation to $\frac{1}{10000}$ in. These instruments magnify the change of length by means of a clockwork device rotating a hand on a dial.

Fig. 4 shows the method of attachment of these extensometers to a column. The gauged length was usually 8 or 9 in. As will be seen, the extensometers measured the change of length between points slightly outside of the members tested. From the actual instrument readings were computed the deformations and stresses at the extreme fibers of the flanges of the channels by the ordinary method, which assumed a rectilinear distribution of deformation and stress.

The type of instrument used is light, simple, and convenient, and, under very severe usage in other tests, it had shown itself to be durable. The limits of accuracy of the instrument were fairly well determined by careful calibration. While the maximum errors possible are large, it gives fairly trustworthy results under careful manipulation, and any available instrument of greater precision would be too bulky or too likely to have its parts deranged under the conditions of column tests, especially in field tests, to be entirely satisfactory.

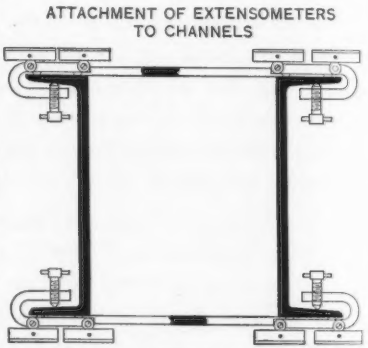

FIG. 4.

The accuracy of all these instruments was tested by comparison with a Brown and Sharpe micrometer acting through a 10 to 1 lever. Basing judgment on the maximum deviation observed in the calibration, and on the smallest deformations measured in any test of columns, it is felt that the error in stress determination for the channel members is generally less than $\pm$ 10%, and that it is well within this limit for most of the determinations in the laboratory tests. This general limit of accuracy is corroborated by a comparison of the average stresses on the center of gravity of the flange members with each other and with the known average stress. To those accustomed to greater precision in calculations and to the greater refinement of most laboratory tests, this limit of accuracy may seem crude. Having in view that the purpose of the tests was qualitative rather than quantitative, and considering the great variation in the distribution of stress found in the columns, and the general consistency of the results, it may be considered that the instruments were satisfactory.

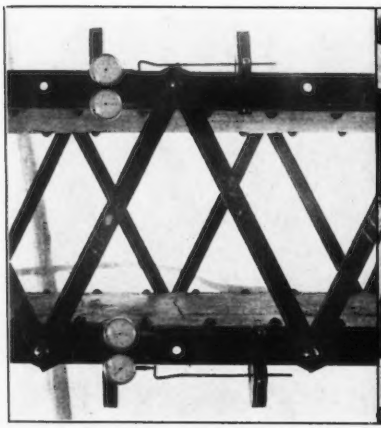

FIG. 1.—EXTENSOMETERS IN PLACE.

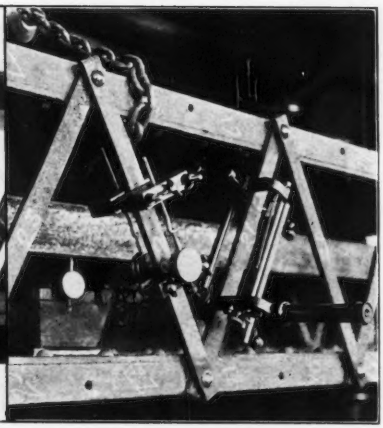

FIG. 2.—ATTACHMENT OF INSTRUMENTS IN CROSS-BENDING TEST OF COLUMN NO. 1.

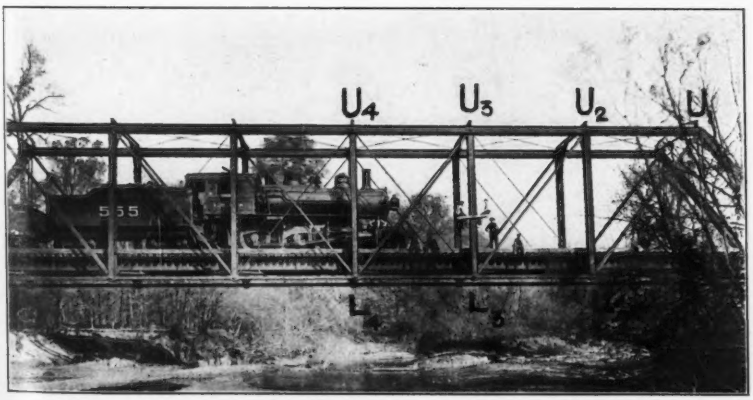

FIG. 3.—WHITE HEATH BRIDGE AND TEST TRAIN.

In several of the tests, both in the laboratory and in the field, extensometers of the Johnson type were also used, but it was found that the results were not trustworthy, and no account of these measurements is given here.

The Ames test gauges were also used to measure deformations in the lattice bars of all columns but No. 1. On account of the very low stresses in the lattice bars, it is felt that the stresses determined in them in this way may be in error by ± 20%, or even more. In Column No. 1, a Ewing extensometer, reading to $\frac{1}{50\,000}$ in. was used, and the stresses in the lattice bars of this column are judged to have been determined with an error in any case of not more than ± 10 per cent.

Testing Machine.—The testing machine used was the Riehle, 600 000-lb., vertical, screw-power testing machine in the Laboratory of Applied Mechanics of the University of Illinois. This machine has a clear space of 36 in. between screws, and will take in compression specimens 25 ft. long. The speed of head used in these tests was in nearly all cases 0.4 in. per minute. This machine is equipped with massive guide frames which take the very heavy side thrust which occurs on the cross-head when a specimen is tested under oblique load. These guide frames are entirely independent of the weighing mechanism.

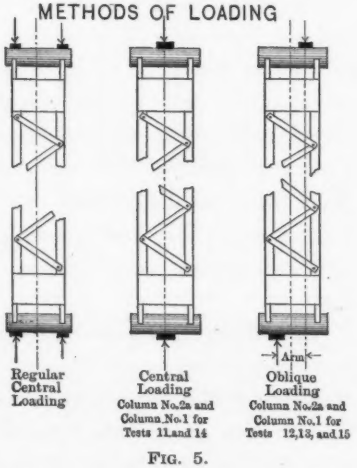

FIG. 5.

Method of Loading.—Two methods of loading were used, central and oblique. In all cases the load was applied at the ends of the column through the pins, and in a plane at right angles to the web and passing through the nominal axis of the column. In central loading, the pin was adjusted to an even bearing on the machine, the object being to apply the load equally to the two webs of the column. In oblique loading, the pin was supported on a narrow block, as shown in Fig. 5, in such a way as to secure a given eccentricity. The center of the block was taken as the point of application of the load. This is approximately true, the error probably not being greater than

¼ in. In two tests with Column No. 1, the point of application of the load is uncertain, as by an oversight the two bearing pieces used on each pin were placed unsymmetrically with respect to the axis of the column. It seems probable that the loading was nearly central.

Routine of Tests for Stress Distribution.—In a test for stress distribution, the column was placed in the machine and a light initial load was applied. The extensometers were then attached in position to measure the deformation at some point of the column, and an initial reading was taken. A known load was applied by the testing machine and the instruments were read again. The load was then released to its initial value and another application of the load was made. If the second readings did not exactly check the first, further applications of the load were made. In cases where the observed deformations were large or seemingly abnormal, the test was repeated at another time, and in some cases as many as ten observations were made on the same gauged length. In some of these cases the instruments were reset, their places being exchanged. The instruments were next attached in a new location, and the process was repeated. Thus the stress distribution in various parts of the column was finally determined. The load generally used in the laboratory tests was 10 000 lb. per sq. in. of section of the column in excess of the initial load.

Results of Tests for Stress Distribution in Channels.—Tables 2 and 3 give results of the tests to determine stress distribution and variation in the flange members found in twelve of the column tests. The stresses given are calculated from the observed deformation, using for the modulus of elasticity 28 000 000 lb. per sq. in. for steel and 26 000 000 lb. per sq. in. for wrought iron, these values checking closely with the total shortening of the columns and with the average deformations observed throughout their length. As heretofore described, the stress noted is the average over a space of 4 or 4½ in. on either side of the point indicated. Any lack of agreement between the average stress on the center of gravity of the flange members and the average stress for the load applied is probably due principally to instrumental errors.

Figs. 6, 7, and 8 show graphically the stress distribution and variation. The full line gives the stress at the west side (front) and the dotted line at the east side (back).

TABLE 2.—Stresses in Column No. 1.

Panel.	North Channel.						South Channel.					
	West side.			East side.			West side.			East side.		
	Outer fiber.	Center of gravity.	Inner fiber.	Outer fiber.	Center of gravity.	Inner fiber.	Outer fiber.	Center of gravity.	Inner fiber.	Outer fiber.	Center of gravity.	Inner fiber.
Test No. 1.												
1	8 100	10 200	10 600	12 200	10 000	9 700	6 500	8 400	8 600	9 600	10 900	11 300
2	9 700	10 100	10 100	8 100	9 600	10 000	11 900	11 500	11 600	8 900	10 500	11 100
3	7 500	10 200	11 000	11 800	10 000	9 700	11 500	9 800	9 800	9 000	10 300	11 000
4	10 500	10 000	10 000	10 500	11 000	11 000	9 000	8 600	8 600	10 800	10 100	10 500
5	9 600	9 100	9 100	10 900	9 700	9 500	10 300	8 500	9 700	8 800	10 100	10 600
6	9 900	11 000	11 100	10 000	9 600	9 700	9 400	8 400	8 100	9 000	9 200	9 100
7	10 000	7 600	6 900	11 000	10 000	9 900	9 000	9 400	9 900	9 200	10 000	10 400
8	8 100	8 900	9 000	10 800	11 300	11 500	12 700	10 500	10 400	12 200	10 100	10 300
9	10 000	11 000	11 100	9 400	11 000	11 000	14 500	10 000	9 000	9 000	9 200	9 200
10	14 000	10 000	9 300	14 300	11 800	11 000	5 400	8 900	9 300	7 200	9 600	10 000
11	8 600	9 500	9 600	10 000	9 800	9 600	9 000	8 800	8 400	9 800	11 000	11 400
12	12 300	9 200	8 700	9 100	9 400	9 800	10 800	9 300	9 000	10 700	9 600	9 500
13	.....	.....	.....	8 700	8 800	9 000	7 800	9 000	9 200	.....	.....	.....
Test No. 2.												
6	11 500	12 100	12 100	11 500	10 000	9 500	9 100	9 000	9 300	5 800	7 500	7 700
7	12 200	12 600	12 700	14 600	13 500	13 300	5 900	7 500	8 000	8 300	9 500	9 500
8	11 300	12 500	12 800	12 400	12 500	10 600	7 800	8 200	8 300	7 500	10 300	10 700
9	14 800	13 300	12 700	9 500	10 500	10 600	11 800	7 300	7 000	10 300	8 400	8 300
10	9 000	10 200	10 400	16 200	10 500	9 600	7 800	10 500	11 400	9 000	8 400	8 500
11	15 200	10 200	9 700	13 700	11 600	11 500	7 600	8 100	8 300	5 000	6 300	8 600
12	10 300	11 900	12 300	12 700	11 600	11 500	9 400	8 200	7 700	7 000	7 500	9 700
13	13 200	11 800	11 700	9 900	10 500	10 800	7 500	8 600	9 100	6 300	6 300	8 400
Test No. 3.												
1	.....	.....	.....	13 800	10 400	9 500	7 500	8 800	8 900	.....	.....	.....
2	8 900	10 600	10 900	8 800	10 400	10 600	8 300	9 300	9 400	10 000	9 700	9 700
3	10 500	9 800	9 800	8 700	9 000	9 200	10 400	8 700	8 700	11 900	10 500	10 300
4	9 800	10 800	11 000	11 100	10 700	10 700	8 300	8 300	8 500	13 300	12 600	12 200
5	10 500	10 000	9 800	11 200	10 700	10 700	9 000	10 000	10 100	9 300	9 500	9 400
6	11 300	12 300	12 500	10 900	10 500	10 700	10 400	9 700	9 500	7 500	9 400	9 600
7	10 400	10 700	10 700	11 900	11 000	10 800	9 200	10 000	10 000	8 400	8 800	8 800
8	12 100	10 200	10 200	12 100	13 000	13 100	8 500	9 300	9 400	7 300	9 000	9 400
9	7 600	9 900	10 200	10 500	10 900	10 800	14 400	9 300	8 400	10 900	10 600	10 700
10	10 500	12 000	12 100	14 200	11 000	10 500	7 200	10 400	10 900	7 800	8 700	8 700
11	14 200	10 500	9 700	7 500	8 500	8 600	8 300	9 400	9 300	7 000	9 400	9 800
12	10 400	12 000	12 200	6 500	7 500	7 800	11 000	9 300	9 000	7 900	9 100	9 400
13	12 600	9 800	9 500	8 300	8 400	8 200	7 200	9 800	10 200	11 000	10 800	10 900

TABLE 2.—STRESSES IN COLUMN NO. 1—(*Continued*).

Panel.	North Channel. West side. Outer fiber.	Center of gravity.	Inner fiber.	North Channel. East side. Outer fiber.	Center of gravity.	Inner fiber.	South Channel. West side. Outer fiber.	Center of gravity.	Inner fiber.	South Channel. East side. Outer fiber.	Center of gravity.	Inner fiber.
Test No. 4.												
1	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
2	11 300	10 800	10 700	10 500	11 100	11 200	6 100	3 000	2 500	8 900	10 200	10 600
3	10 000	11 600	11 600	10 100	11 100	11 100	8 400	8 500	8 500	12 000	11 300	11 100
4	11 200	12 300	12 500	9 900	10 900	11 000	7 600	9 400	9 700	7 900	9 300	9 500
5	.....	.....	.....	12 800	11 100	10 700	.....	.....	.....	10 700	10 200	10 100
6	10 100	5 400	4 500	12 800	10 700	10 300	6 500	8 900	9 500	.....	.....	.....
7	8 000	10 600	11 000	10 200	9 700	9 700	.....	.....	.....	8 700	8 500	8 600
8	9 800	12 000	12 400	9 700	10 000	10 100	9 000	9 400	9 400	10 300	11 300	11 400
9	11 500	10 600	10 400	.....	.....	.....	8 500	10 700	10 900	7 900	11 100	11 500
10	8 600	8 300	8 100	6 800	8 700	9 100	10 800	7 500	7 000	9 300	10 700	11 000
11	.....	.....	.....	.....	.....	.....	9 500	9 000	8 900	13 000	12 700	12 600
12	10 900	10 400	10 300	10 700	9 900	9 600	.....	.....	.....	.....	.....	.....
13	11 000	10 700	10 700	11 100	10 600	10 500	7 300	9 800	9 600	12 700	11 900	11 800
Test No. 5.												
½	12 100	10 400	9 300	11 000	9 500	8 700	9 700	10 500	10 900	10 400	10 800	10 900
1	9 800	11 100	11 800	10 000	10 200	10 500	12 200	10 400	9 300	13 500	11 300	10 400
1½	7 900	8 500	8 600	5 800	8 200	8 900	7 300	8 400	8 900	7 400	7 900	8 300
2	11 800	11 400	11 100	10 900	10 600	10 600	8 500	8 600	8 700	9 800	10 200	10 400
2½	10 700	11 000	11 100	10 800	10 200	10 200	11 800	11 900	11 800	9 700	9 700	9 600
3	5 800	7 500	8 500	7 800	8 000	8 500	8 300	8 500	8 600	7 700	9 100	9 700
3½	8 000	8 100	8 500	7 400	8 100	8 500	7 800	8 600	9 000	7 500	7 600	7 700
4	15 400	13 000	12 400	13 500	13 000	12 800	11 600	11 800	11 800	9 900	9 900	9 900
4½	12 200	10 700	10 300	12 200	10 200	9 200	9 900	9 900	9 800	9 800	9 800	9 700
5	9 400	9 500	9 600	11 000	10 100	9 600	9 700	9 500	9 500	8 900	9 800	9 900
5½	9 900	10 700	11 100	10 500	10 900	11 000	10 900	9 100	8 300	8 700	8 800	8 900
6	7 400	7 300	7 200	8 100	9 200	9 700	13 800	11 900	11 100	11 100	10 600	10 200
6½	10 900	10 200	9 700	10 200	10 300	10 500	8 600	8 900	8 900	8 500	8 600	8 600
7	12 100	11 700	11 400	11 400	10 900	10 400	8 300	8 800	8 900	5 900	7 900	8 700
7½	9 900	9 200	8 900	9 600	9 400	9 500	6 500	7 600	8 300	4 600	6 400	6 900
8	9 800	8 600	8 300	8 100	9 700	10 400	11 400	9 500	8 500	12 700	10 800	10 100
8½	9 200	9 700	10 000	9 700	9 500	9 300	20 200	15 400	13 100	15 400	14 000	13 200
9	6 500	7 600	8 300	9 700	9 700	9 900	12 900	13 200	13 100	9 400	9 900	10 200
9½	10 000	10 400	10 600	12 900	9 500	8 100	4 600	7 800	8 900	4 600	7 300	8 500
10	16 400	12 900	11 200	17 000	14 400	12 900	8 000	8 600	8 700	6 800	8 000	8 600
10½	11 300	10 700	10 400	9 900	10 000	9 700	10 500	11 100	11 900	7 400	8 900	9 600
11	7 000	8 400	8 900	8 100	9 100	9 600	12 600	11 100	10 800	9 800	10 800	11 000
11½	9 700	8 900	8 600	11 900	10 500	9 700	8 200	11 100	11 700	8 900	9 700	9 800
12	14 000	12 400	11 600	11 900	11 000	10 400	6 900	7 500	7 800	8 100	8 800	9 400
12½	9 700	8 900	8 600	10 500	12 100	12 700	10 800	11 800	12 400	11 900	11 000	10 800

TABLE 3.—Stress in Wrought-Iron Bridge Posts.

Panel.	North Channel. West side. Outer fiber.	Center of gravity.	Inner fiber.	North Channel. East side. Outer fiber.	Center of gravity.	Inner fiber.	South Channel. West side. Outer fiber.	Center of gravity.	Inner fiber.	South Channel. East side. Outer fiber.	Center of gravity.	Inner fiber.
Column No. 2, Test No. 6.												
1	4 600	6 500	7 300	6 400	7 900	8 500	4 300	8 300	9 700	5 000	6 800	7 400
2	7 300	7 300	7 300	8 300	8 400	8 400	9 800	9 000	8 800	9 000	8 000	7 600
3	5 500	5 900	6 000	7 100	7 800	8 000	7 700	8 800	9 200	6 700	7 600	7 900
4	6 200	6 100	6 200	6 100	6 900	7 100	8 400	8 800	9 000	8 200	8 000	8 000
5	7 200	6 500	6 100	8 800	8 600	8 500	7 500	8 800	9 300	8 700	7 800	7 500
6	6 800	6 900	6 900	8 400	8 400	8 400	7 100	7 600	7 800	6 900	7 200	7 400
7	7 500	7 900	8 000	7 100	7 400	7 500	7 500	8 200	8 500	7 500	7 400	7 300
8	6 430	7 400	7 800	6 200	7 200	7 600	7 700	7 900	8 000	6 600	6 400	6 100
Column No. 3, Test No. 7.												
1	9 300	9 000	9 000	8 400	9 500	10 000	10 000	9 000	10 100	8 200	8 000	7 900
2	10 200	10 000	9 900	10 200	11 500	11 900	8 400	9 100	10 000	9 100	8 700	8 500
3	8 800	8 700	8 600	9 800	10 000	10 000	10 000	9 700	9 600	9 400	9 600	9 600
4	10 500	9 100	8 600	8 500	9 500	9 900	8 600	9 800	10 200	9 300	7 500	6 900
5	10 200	10 400	10 500	8 800	9 600	9 900	10 100	9 800	9 700	10 400	10 200	10 200
6	10 600	10 000	9 800	10 300	10 000	10 000	10 400	10 400	10 400	10 000	9 900	9 900
7	10 300	9 900	9 700	9 600	9 900	10 000	8 700	9 500	9 700	10 900	10 000	9 700
8	9 800	9 600	9 500	10 200	10 900	11 000	7 600	9 600	10 300	7 800	9 900	10 600
Column No. 4, Test No. 8.												
1	9 000	9 800	10 000	10 000	10 700	11 000	8 200	10 000	10 800	9 000	9 200	9 300
2	8 700	9 300	9 500	10 100	10 800	11 100	10 100	11 600	12 100	8 600	9 400	9 700
3	9 300	9 800	9 900	10 800	11 200	11 400	10 000	10 200	10 200	7 400	8 400	8 700
4	9 500	9 700	9 800	11 100	10 200	10 000	9 700	11 100	11 600	7 100	8 200	8 500
5	10 200	10 500	10 600	11 900	12 400	12 600	8 400	9 400	9 800	7 700	8 800	9 200
6	11 600	12 000	12 100	12 400	13 600	13 900	9 500	10 800	11 100	7 400	9 500	10 200
7	12 800	12 500	12 400	13 400	14 200	14 600	7 100	8 300	8 700	6 400	6 700	6 700
8	8 600	11 600	12 700	13 000	11 300	10 700	6 600	8 500	9 200	6 200	7 800	8 400
Column No. 5, Test No. 9.												
½	14 000	11 700	10 800	12 600	11 700	11 400	7 400	8 100	8 400	7 400	7 900	8 000
1	11 300	12 000	12 300	11 100	10 000	9 600	6 700	8 500	9 200	5 100	6 800	7 500
1½	11 300	11 100	11 100	11 100	10 800	10 700	5 700	6 200	6 300	6 600	6 900	7 100
2	13 100	12 600	12 400	12 900	12 400	12 200	8 000	8 000	8 000	7 800	8 200	8 300
2½	12 700	12 000	11 700	13 100	11 500	11 100	9 300	9 200	9 200	8 700	8 400	8 300
3	11 800	12 000	12 200	11 400	12 200	12 400	8 200	8 400	8 400	6 300	8 000	8 600
3½	13 500	10 500	9 300	11 600	10 700	10 400	8 200	8 200	8 200	8 400	8 300	8 200
4	14 300	13 100	12 700	14 300	12 300	11 500	7 300	7 400	7 500	7 400	8 700	9 200
4½	11 600	11 600	11 500	12 300	12 300	12 300	9 000	9 000	9 000	8 700	8 700	7 800
5	12 600	12 400	12 300	11 700	10 400	10 000	8 300	8 400	8 400	7 400	8 500	8 100
5½	13 700	12 100	11 500	13 500	12 100	11 400	8 300	8 300	8 300	8 400	8 400	7 600
6	12 600	12 700	12 700	13 500	12 500	12 200	7 600	7 600	7 600	7 200	7 000	7 100
6½	13 100	11 900	11 500	11 400	11 400	11 500	8 400	8 400	8 400	10 500	9 000	8 500
7	11 100	11 300	11 500	11 200	12 300	12 600	7 900	8 000	8 000	6 500	8 100	8 600
7½	10 600	10 100	9 900	11 000	10 300	9 900	8 000	8 200	8 200	5 900	7 900	8 700
8	10 300	12 300	12 800	11 300	12 300	12 400	8 500	8 600	8 600	10 800	11 800	12 200

TABLE 3—(*Continued*).

Panel.	North Channel. West side. Outer fiber.	Center of gravity.	Inner fiber.	East side. Outer fiber.	Center of gravity.	Inner fiber.	South Channel. West side. Outer fiber.	Center of gravity.	Inner fiber.	East side. Outer fiber.	Center of gravity.	Inner fiber.
Column No. 4*a*, Test No. 10.												
½	8 100	9 500	10 200	8 400	8 100	8 000	11 700	10 800	10 500	9 900	11 200	11 800
1	8 300	10 200	10 800	6 500	7 200	7 500	12 100	10 900	10 400	11 100	12 500	13 100
1½	8 400	9 400	9 700	7 400	8 200	8 400	12 100	11 400	11 100	11 000	11 400	11 600
2	8 700	8 800	9 000	6 200	7 200	7 500	10 800	10 500	9 700	9 300	11 700	12 600
2½	8 300	8 800	9 000	7 300	6 900	6 800	11 800	11 500	12 100	10 900	10 400	10 300
3	8 700	9 700	9 900	6 700	6 500	6 400	13 300	11 100	10 400	10 400	11 100	11 400
3½	8 300	8 000	8 000	7 300	7 300	7 400	13 600	11 600	11 100	13 000	12 400	12 400
4	8 200	9 700	10 100	6 800	6 900	6 800	14 300	13 500	12 700	13 100	11 100	10 500
4½	7 500	8 600	9 100	6 300	7 200	7 500	12 300	11 100	10 800	10 300	15 200	9 400
5	7 500	8 400	8 800	7 000	8 100	8 500	13 000	13 300	13 500	9 000	11 400	12 300
5½	6 300	7 200	7 500	5 900	6 100	6 200	12 700	12 300	12 100	12 300	10 700	10 100
6	6 800	7 800	8 200	5 300	6 800	7 200	14 200	14 200	14 200	11 000	11 200	11 300
6½	6 600	7 800	8 200	7 100	7 100	7 100	15 400	13 700	12 900	13 500	11 500	11 100
7	6 700	7 100	7 200	4 900	6 100	6 700	11 900	13 600	14 600	11 100	12 100	12 400
7½	6 100	7 600	8 300	4 800	8 100	9 400	12 300	12 400	12 400	9 300	16 200	11 100
8	6 000	8 200	9 000	8 400	6 800	6 200	10 700	12 200	12 800	10 000	16 300	11 100
Column No. 2*a*, Test No. 11.												
½	11 600	10 900	10 400	11 400	10 500	10 000	.....	.....	.....	.....	.....	.....
1	7 300	9 300	10 000	7 300	9 700	10 400	8 400	8 800	9 000	10 200	10 200	10 200
1½	11 600	9 900	9 600	11 600	10 900	10 000	8 500	10 000	10 600	11 400	12 000	12 200
2	8 900	8 700	8 600	10 500	10 300	10 000	10 200	9 000	8 600	11 500	11 200	11 000
2½	11 200	9 700	9 200	10 900	10 300	10 000	9 900	10 000	10 200	11 700	10 300	9 900
3	9 500	10 500	10 700	12 200	10 800	10 000	11 800	10 200	10 800	11 400	10 600	10 300
3½	10 500	11 000	11 000	12 100	10 600	10 000	10 400	8 600	8 100	9 700	9 600	9 400
4	9 800	9 500	9 400	10 600	9 900	9 600	9 800	9 400	9 200	10 400	10 600	10 500
4½	10 300	9 600	9 300	11 200	10 200	10 000	8 700	9 600	10 400	10 400	10 000	10 000
5	9 600	9 800	9 900	10 400	10 800	11 000	10 300	9 200	8 800	10 400	10 200	10 200
5½	12 200	11 600	11 200	12 000	11 100	10 700	12 500	10 400	10 000	12 200	11 900	11 700
6	10 100	9 700	9 700	9 900	9 700	9 700	9 100	10 400	10 900	10 300	10 700	10 700
6½	11 300	11 000	10 800	10 600	9 900	9 700	10 800	9 900	9 700	11 000	11 000	10 000
7	9 300	9 600	9 700	9 100	9 300	9 200	9 800	9 800	9 800	9 800	10 800	10 000
7½	10 200	10 900	11 400	9 100	9 400	9 500	9 300	9 600	9 800	9 900	10 700	10 800
8	9 900	9 800	9 700	11 100	10 000	9 600	8 800	10 300	10 900	10 800	10 700	10 800
Column No. 2*a*, Test No. 12.												
½	10 800	10 000	9 800	10 400	10 600	10 600	11 500	10 600	10 000	13 200	12 600	12 300
1	8 400	8 400	8 500	9 700	10 800	11 200	9 800	10 700	10 900	10 000	11 300	11 700
1½	8 200	8 000	7 900	9 200	10 600	10 900	9 200	9 600	9 700	8 800	10 000	10 200
2	9 200	9 800	10 000	9 900	10 900	11 300	11 400	10 000	9 400	12 200	11 400	11 000
2½	10 800	9 900	9 600	10 800	9 600	9 100	10 900	10 900	10 900	11 400	11 600	11 600
3	9 100	10 800	11 200	11 000	10 600	10 400	10 600	11 200	11 300	10 400	10 200	10 100
3½	10 700	10 900	10 800	12 000	10 600	10 000	10 600	10 900	10 900	10 200	10 600	10 000
4	10 000	8 800	8 400	11 400	9 400	9 400	8 900	10 000	10 300	8 800	10 300	10 900
4½	10 600	10 000	9 900	10 200	10 000	9 900	10 000	10 100	10 200	8 600	9 500	9 800
5	10 100	10 300	10 400	11 300	11 000	11 000	11 800	10 200	9 600	10 200	9 700	9 700
5½	10 900	11 300	11 300	11 400	10 600	10 300	13 000	11 000	10 400	11 000	9 500	9 000
6	11 500	10 800	10 500	9 500	9 000	9 000	9 500	9 900	10 000	9 700	9 500	9 500
6½	11 700	9 800	9 200	12 700	10 300	9 600	10 000	10 000	10 000	9 600	10 300	10 600
7	9 900	11 300	11 600	9 900	11 100	10 600	16 500	9 800	9 300	9 600	10 000	10 000
7½	8 000	9 700	10 300	15 400	15 400	15 400	7 800	10 000	10 600	6 800	8 500	9 000
8	10 000	9 800	9 900	11 700	10 100	9 900	8 800	10 400	11 100	9 400	9 400	9 400

TABLE 3—(*Continued*).

Panel.	North Channel.						South Channel.					
	West side.			East side.			West side.			East side.		
	Outer fiber.	Center of gravity.	Inner fiber.	Outer fiber.	Center of gravity.	Inner fiber.	Outer fiber.	Center of gravity.	Inner fiber.	Outer fiber.	Center of gravity.	Inner fiber.
COLUMN No. 2*a*, TEST No. 13.												
1	4 500	7 000	8 600	7 000	8 600	9 100	9 800	10 800	11 100	10 300	12 300	13 000
2	7 600	9 000	9 500	10 300	10 400	10 600	10 900	10 100	9 700	12 800	12 600	12 600
3	7 000	8 600	9 100	9 400	10 100	10 300	11 300	11 300	11 300	11 600	12 600	13 000
4	8 100	9 100	9 500	10 400	10 800	10 900	9 300	10 400	10 700	9 600	11 600	12 500
5	8 200	9 500	10 000	10 000	10 200	10 200	10 900	8 900	8 100	10 600	10 700	10 800
6	10 000	10 400	10 600	10 500	10 000	10 000	8 500	9 600	10 000	9 600	10 400	10 800
7	9 800	9 800	9 900	10 200	10 300	10 300	9 500	8 800	8 700	9 900	10 000	10 100
8	8 000	7 800	7 700	12 300	10 300	9 200	7 700	8 700	9 100	8 700	10 000	10 500

Table 4 gives a number of the most marked deviations from average stress. The excess of the maximum fiber stress is given as a percentage of the average stress.

TABLE 4.—MAXIMUM OBSERVED FIBER STRESSES IN FLANGE MEMBERS OF COLUMNS.

Column Number.	Test Number.	Features of Lacing.	Method of Loading.	Percentage of excess of maximum fiber stress over average stress. Highest five values.
1	1	1¼ by $\frac{7}{16}$ in., bolted.....	Central.	42, 41, 39, 31, 23
1	2	"	Slightly eccentric.	68, 64, 50, 49, 35
1	3	"	"	50, 41, 35, 32, 19
1	4	1 by ½ in., bolted.....	Central.	35, 29, 28, 27, 26
1	5	1 by ½ in	"	67, 55, 49, 29, 27
2	6	.....	"	31, 23, 23, 21, 17
3	7	.....	"	20, 17, 12, 11, 9
4	8	.....	"	41, 29, 24, 22, 19
5	9	.....	"	43, 43, 38, 35, 35
4a	10	.....	"	53, 49, 42, 40, 37
2a	11	.....	"	21, 16, 13, 12, 11
2a	12	.....	Oblique arm, 1 in. (Fig. 5).	42, 42, 24, 23, 20
2a	13	.....	Oblique arm, 2 in. (Fig. 5).	35, 34, 22, 21, 21

In most cases the maximum stress was in the outer fiber of the channel; sometimes very high stresses were found in the inner fiber. Generally, the stress in the opposite channel was correspondingly less.

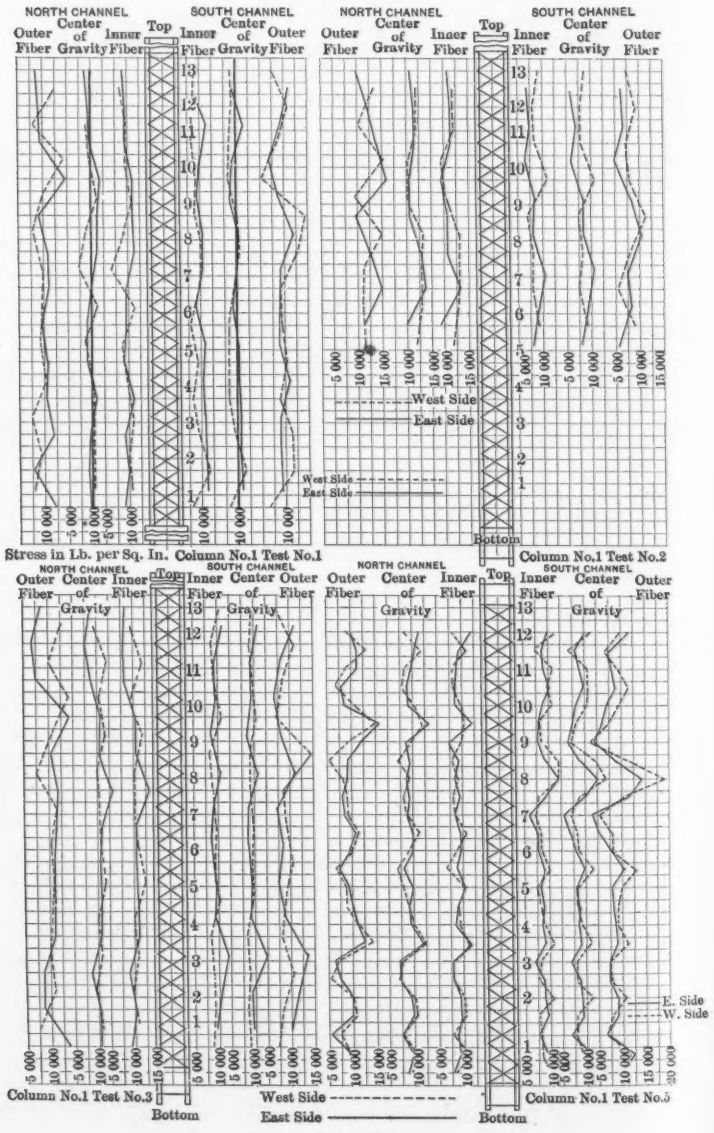
STRESS DISTRIBUTION IN CHANNELS OF COLUMN NO. 1
NORTH CHANNEL
Outer Fiber
Center of Gravity
Inner Fiber
Top
SOUTH CHANNEL
Inner Fiber
Center of Gravity
Outer Fiber
West Side
East Side
Bottom
Stress in Lb. per Sq. In.
Column No.1 Test No.1
Column No.1 Test No.2
Column No.1 Test No.3
Column No.1 Test No.5
E. Side
W. Side

FIG. 6.

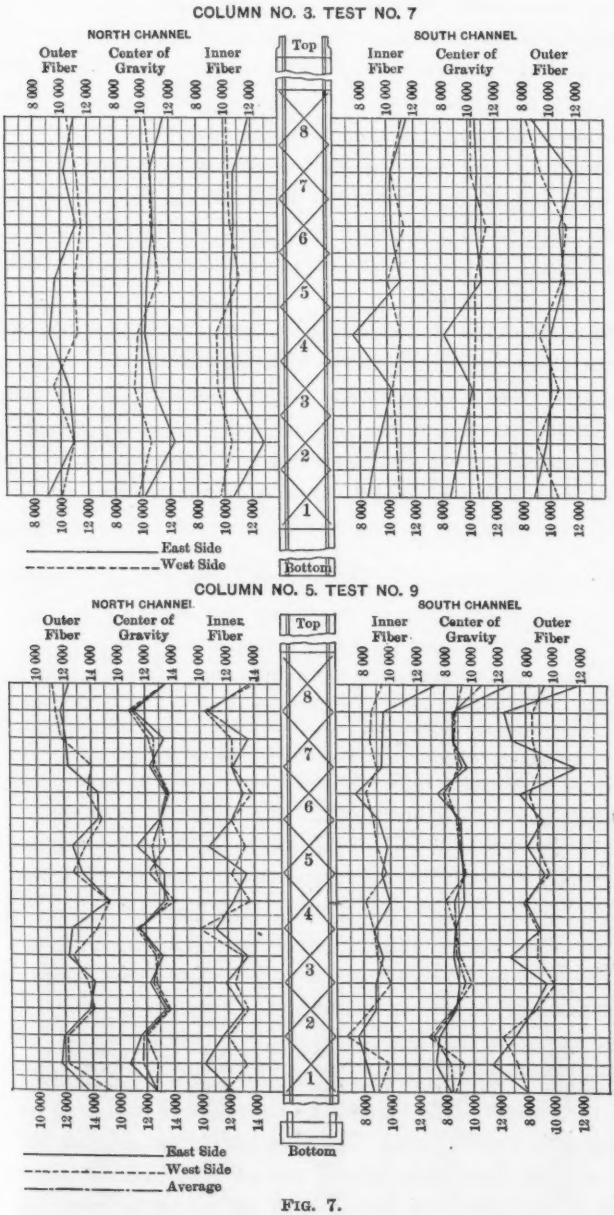
COLUMN NO. 3. TEST NO. 7
NORTH CHANNEL
Outer Fiber
Center of Gravity
Inner Fiber
SOUTH CHANNEL
Inner Fiber
Center of Gravity
Outer Fiber
Top
Bottom
East Side
West Side
COLUMN NO. 5. TEST NO. 9
NORTH CHANNEL
SOUTH CHANNEL
Average

FIG. 7.

Stress in Lattice Bars.—Table 5 gives the results of tests to determine the average stress in the various lattice bars of the columns. Tests 14 and 15 were tests on the lattice bars only. The distribution of stress over the cross-section of the bar is discussed in another place. The average stresses in the lattice bars are computed from the observed deformations, using a modulus of elasticity of 28 000 000 lb. per sq. in. for the steel column and 26 000 000 lb. per sq. in. for the wrought-iron columns. As might be expected, from the irregular variation of stress along the flange members of the columns, the stress in the lattice bars was found to vary greatly.

Table 6 gives the largest stresses observed and the corresponding transverse shear. The transverse shear given in this table is that which would cause a stress in the lattice bars equal to the maximum stress observed in any lattice bar, and was computed by doubling the transverse component of the maximum load observed on a lattice bar. In the case of obliquely loaded columns, the transverse component of the load was computed on the assumption that the load was applied

TABLE 5.—TOTAL STRESS, IN POUNDS, ON LATTICE BARS UNDER LOAD ON COLUMNS OF 10 000 LB. PER SQ. IN.

COLUMN NO. 1.

Lattice bar.	Test No. 5.		Test No. 14.		Test No. 15.	
	Front.	Back.	Front.	Back.	Front.	Back.
1	400*c*	400*c*	300*t*	400*t*	.....	.....
2	100*t*	600*t*	1 600*t*	400*c*	.....	.....
3	0	600*c*	800*c*	400*t*	.....	.....
4	100*t*	400*t*	300*t*	100*c*	.....	.....
5	100*c*	100*c*	.....	.....	.....	.....
6	200*t*	300*t*	.....	.....	.....	.....
7	100*c*	200*c*	.....	.....	.....	.....
8	200*t*	200*t*	.....	.....	.....	.....
9	0	300*c*	.....	.....	.....	.....
10	0	200*t*	.....	.....	.....	.....
11	100*c*	300*c*	.....	.....	.....	.....
12	0	100*t*	.....	.....	.....	.....
13	100*c*	200*c*	800*c*	0	.....	.....
14	100*t*	200*t*	900*t*	900*t*	2 500*t*	1 600*c*
15	0	300*c*	200*c*	1 000*t*	2 100*c*	.....
16	0	400*t*	800*t*	0	1 600*t*	200*c*
17	200*c*	200*c*	500*c*	300*t*	1 900*c*	.....
18	200*t*	100*t*	700*t*	1 000*t*	700*t*	1 100*t*
19	500*c*	200*c*	500*c*	200*t*	1 500*c*	.....
20	200*t*	300*t*	800*c*	200*t*	1 400*t*	700*t*
21	0	600*c*	800*c*	600*t*	1 500*c*	0
22	100*c*	2 000*t*	0	200*t*	700*t*	300*t*
23	200*t*	900*c*	300*c*	800*c*	1 300*c*	.....
24	200*c*	800*t*	200*t*	800*t*	600*t*	400*c*
25	200*t*	800*c*	0	0	2 700*c*	.....
26	200*c*	700*t*	400*t*	300*c*	400*t*	400*c*

TABLE 5—(*Continued*).

COLUMN No. 2*a*, TEST No. 11.

Lattice bar.	FRONT SIDE.		BACK SIDE.	
	Under.	Over.	Under.	Over.
1	700*c*	700*c*	1 300*c*	100*c*
2	100*c*	100*t*	200*t*	200*t*
3	3 000*t*	0	1 000*t*	0
4	100*c*	100*c*	800*c*	200*c*
5	3 000*t*	0	800*c*	300*c*
6	200*t*	200*c*	0	300*c*
7	3 700*t*	0	200*t*	200*c*
8	0	800*c*	800*c*	300*c*

COLUMN No. 2*a*, TEST No. 12.

1	1 500*c*	2 100*t*	200*c*	200*c*
2	200*c*	400*t*	2 300*t*	200*c*
3	800*t*	200*t*	2 750*t*	400*c*
4	700*c*	500*t*	2 750*t*	300*c*
5	1 600*t*	200*t*	2 750*t*	300*c*
6	0	0	2 800*t*	800*c*
7	1 400*t*	800*c*	4 100*t*	300*c*
8	100*t*	800*c*	300*t*	800*c*

COLUMN No. 2*a*, TEST No. 13.

1	1 000*c*	700*t*	800*t*	500*c*
2	500*c*	700*t*	2 650*t*	400*c*
3	700*c*	900*t*	2 750*t*	300*c*
4	500*c*	1 000*t*	2 100*t*	200*t*
5	800*c*	600*t*	3 150*t*	1 000*c*
6	1 000*c*	700*t*	4 100*t*	700*c*
7	700*c*	900*t*	1 500*t*	400*c*
8	1 600*c*	200*c*	2 000*t*	1 400*c*

through the center of the bearing blocks. This transverse component was then subtracted from the amount of shear which had been calculated from the deformation of the lattice bars as before noted, and the remainder has been tabulated under the heading "Transverse shear in column due to nominal central load."

Tests to Failure.—After the wrought-iron bridge posts had been tested for stress distribution under working loads, they were loaded to failure. Deformations were measured in the flange members of that part of the column on which the previous test had given the heaviest stress. Table 7 gives the result of the tests to failure. For all the tests of wrought-iron bridge posts, whether loaded centrally or eccentrically, the failures were very gradual. Final failure occurred near the middle or at the end. In the former case, high stresses in one channel had been shown by the deformation measurements at working

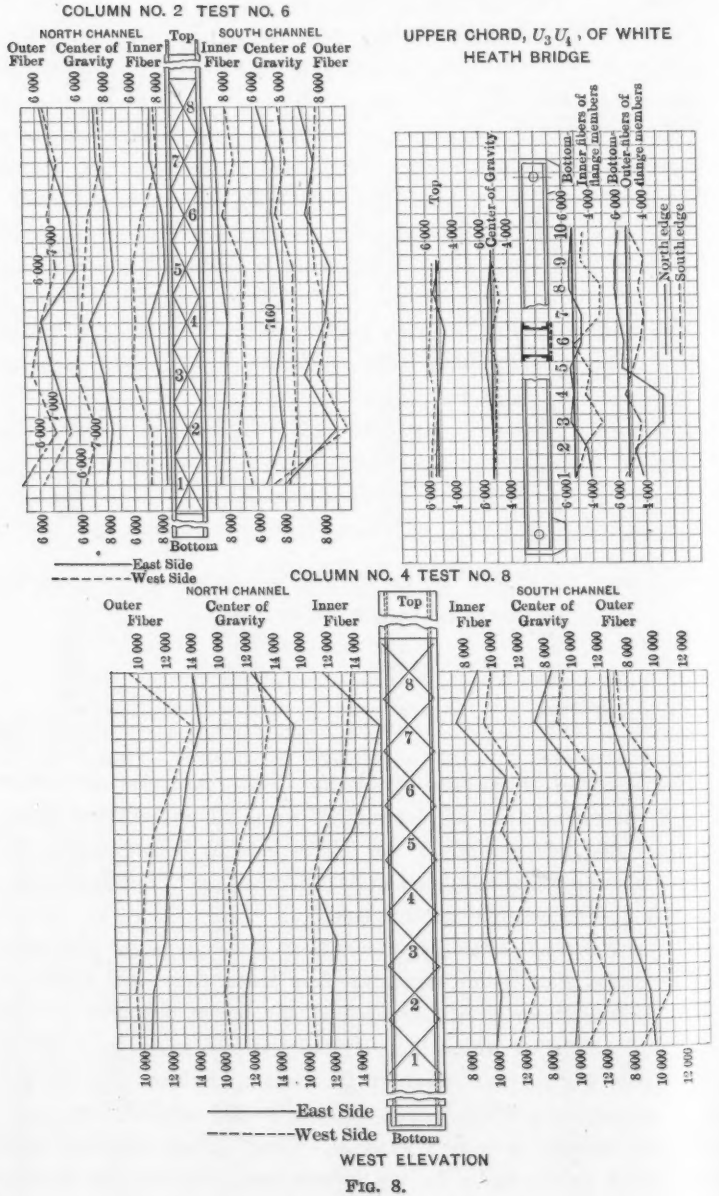
COLUMN NO. 2 TEST NO. 6
NORTH CHANNEL
Outer Fiber
Center of Gravity
Inner Fiber
Top
SOUTH CHANNEL
Inner Fiber
Center of Gravity
Outer Fiber
Bottom
East Side
West Side
UPPER CHORD, U_3U_4, OF WHITE HEATH BRIDGE
Top
Center of Gravity
Bottom
Inner fibers of flange members
Outer fibers of flange members
North edge
South edge
COLUMN NO. 4 TEST NO. 8
NORTH CHANNEL
Outer Fiber
Center of Gravity
Inner Fiber
Top
SOUTH CHANNEL
Inner Fiber
Center of Gravity
Outer Fiber
East Side
West Side
Bottom
WEST ELEVATION

FIG. 8.

TABLE 6.—MAXIMUM OBSERVED AVERAGE STRESS IN LATTICE BARS OF COLUMNS NOS. 1 AND 2*a*.

Column number	1	1	1	2*a*	2*a*	2*a*
Test number	5	14	15	11	12	13
Method of loading	Central.	Central.	Oblique arm, 4 in. (Fig. 5).	Central.	Oblique arm, 1 in. (Fig. 5).	Oblique arm, 2 in. (Fig. 5).
Total load, in pounds	187 600	187 600	187 600	176 400	176 400	176 400
Lacing	Single, 63° 30′ with axis, 1 by ¼ in., riveted.	Single, 63° 30′ 1¼ by $\frac{7}{16}$ in., riveted.	Single, 63° 30′ 1¼ by $\frac{7}{16}$ in., riveted.	Double, 45° 2½ by ⅜ in., riveted.	Double, 45° 2½ by ⅜ in., riveted.	Double, 45° 2½ by ⅜ in., riveted.
Maximum observed average stress in lattice bar, in pounds per square inch	8 200	3 000	4 900	3 900	4 350	4 350
Corresponding total stress on bar, in pounds	2 000	1 600	2 700	3 700	4 100	4 100
Corresponding transverse shear in column, in pounds	3 700	2 900	4 8 0	5 200	5 800	5 800
Shear due to known eccentricity of load, in pounds	0	0	3 000	0	1 000	2 000
Transverse shear in column due to nominal central load, in pounds	3 700	2 900	1 800	5 200	4 800	3 800
Ratio of shear to compression load	0.020	0.016	0.009	0.029	0.027	0.021
Next highest observed values of ratio of shear to compressive load	0.009 0.008 0.008 0.007	0.010 0.010 0.009 0.009	0.009 0.007 0.006 0.006	0.024 0.023 0.010 0.008	0.019 0.018 0.018 0.018	0.016 0.014 0.014 0.011

loads. In the latter case, a bending in one channel at working loads was noted by the instruments at the panel nearest the end of the column. In two of the three columns in which failure took place in the end of the column, as the instruments did not show over-stress in the laced portion of the column, the injured ends were removed, new end connections were put on, and the columns were retested as Nos. 2*a* and 4*a*.

TABLE 7.—RESULTS OF TESTS TO FAILURE.

Column Number..	2	3	4	5	2*a*	4*a*
Test number.....	6	7	8	9	13	10
Method of loading	Central.	Central.	Central.	Central.	Oblique arm. 2 in. (Fig. 5).	Central.
Load at failure, in pounds.....	466 000	480 000	452 000	492 000	475 000	475 000
Average stress at failure, in pounds per square inch..	26 400	27 200	25 700	27 900	26 900	26 900
Average stress at failure in tests of short pieces of flange members, in pounds per inch.....	36 800	36 800	36 800	36 800	36 800	36 800
Percentage of excess of ultimate strength of short pieces over column strength.	39.5	35.5	43.5	32.0	37.0	37.0
Load at first sign of yielding, in pounds	317 000	264 000	256 000	264 000	.....	.....
Average stress at first sign of yielding, in pounds per square inch.	18 000	15 000	14 500	15 000	.....	.....
Method of failure, and remarks....	End failed.	End failed.	End failed.	Bowed at middle in plane perpendicular to lacing.	Buckled near bottom in plane parallel to lacing.	Bowed at middle in plane perpendicular to lacing.

Failure by Buckling of Lattice Bars.—In one of the tests of Column No. 1, the lattice bars failed by buckling suddenly and without warning. As tested, the column was fitted with the light lattice bars (1 by ¼ in.) riveted in place. The test had in view the trial for stress distribution under a slight obliquity, which was not carefully determined. No measuring instruments were in place. A preliminary load was being applied. When the load reached 150 000 lb. (8 060 lb. per sq. in. of cross-section), the alternate lattice bars in the upper half of the column buckled. A failure of this kind was quite unexpected at such a low load. Although an observer was watching the column, the failure was so sudden that he was unable to follow the movement of the parts. In this respect it was quite in contrast to the

failure of the other columns. The machine was at once stopped. Little damage was done to the column, except to the lacing bars. The webs were easily straightened, new lacing bars put on, and the column was used in another test.

Tests to destruction under compression had previously been made on lattice bars like those used in this column, and the results, in the absence of other data, may be useful in estimating the load carried by the lattice bars at failure. Under conditions of loading similar to the conditions found in column lattice bars, these sample bars failed under an average load of 2 100 lb. Assuming that the bar in this column which first failed was carrying 2 100 lb. when failure occurred, the transverse shear in the column may be computed. The following is a tabulated statement of the results of this test:

Column.	Compressive load, in pounds.	Lacing.	Manner of loading.	Probable maximum load on lattice bar, in pounds.	Corresponding shear in column, in pounds.	Ratio of transverse shear to compression load.
No. 1.....	150 000	Single 63° 30′ 1 by ¼, riveted.	Very slight obliquity.	2 100	3 760	0.0251

The column, now riveted up with the heavier lacing bars (1¼ by $\frac{7}{16}$ in.), was loaded obliquely, as shown in Fig. 5, with 300 000 lb. (15 900 lb. per sq. in. of cross-section), and the stresses in the lacing bars were measured. No sign of failure was apparent, and, from the data given in a succeeding paragraph, it appears that under this load the lattice bars must have been stressed to about three-quarters of their ultimate strength in compression.

Cross-Bending Tests of Columns.—Cross-bending tests were made on one of the wrought-iron columns and on Column No. 1. The tests were made in an Olsen 200 000-lb. testing machine fitted for testing beams 20 ft. long. The columns were supported at the ends and loaded at the center with a light transverse load. The column was placed first with the plane of the lacing perpendicular to the load, and then with the plane of the latticing parallel to the load. The lattice bars used in the tests of Column No. 1 were 1¼ by $\frac{7}{16}$ in. in cross-section; in one test they were bolted in place and in another they were riveted. The deflection at various points along the beam was measured

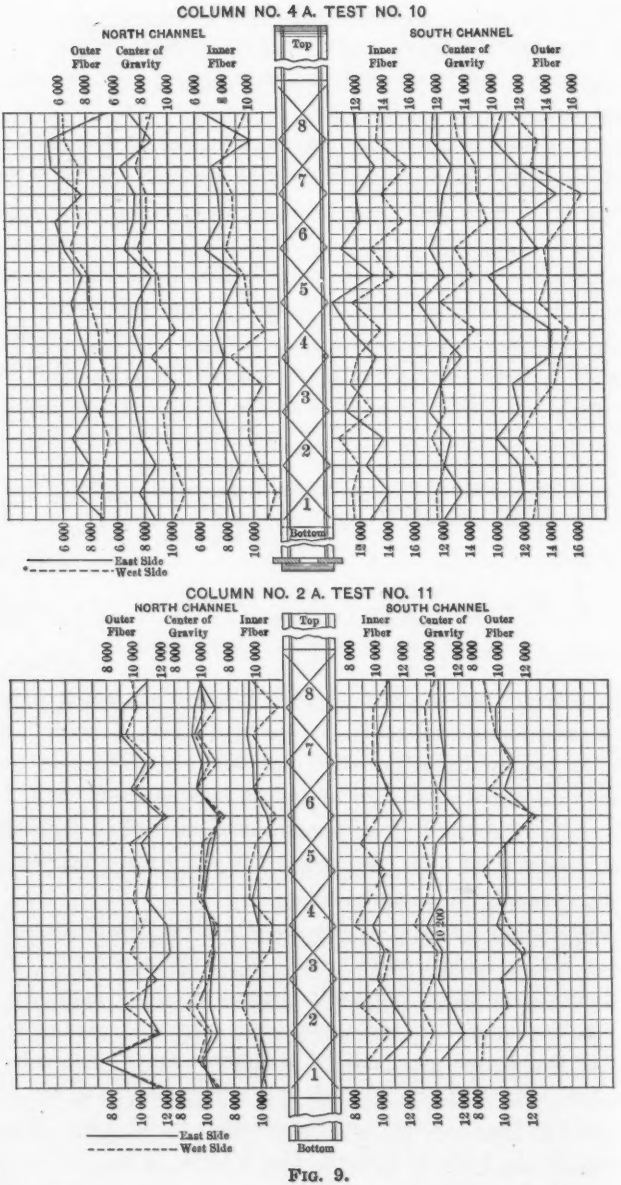
COLUMN NO. 4 A. TEST NO. 10
NORTH CHANNEL
Outer Fiber
Center of Gravity
Inner Fiber
Top
SOUTH CHANNEL
Inner Fiber
Center of Gravity
Outer Fiber
Bottom
East Side
West Side
COLUMN NO. 2 A. TEST NO. 11
NORTH CHANNEL
SOUTH CHANNEL
East Side
West Side

FIG. 9.

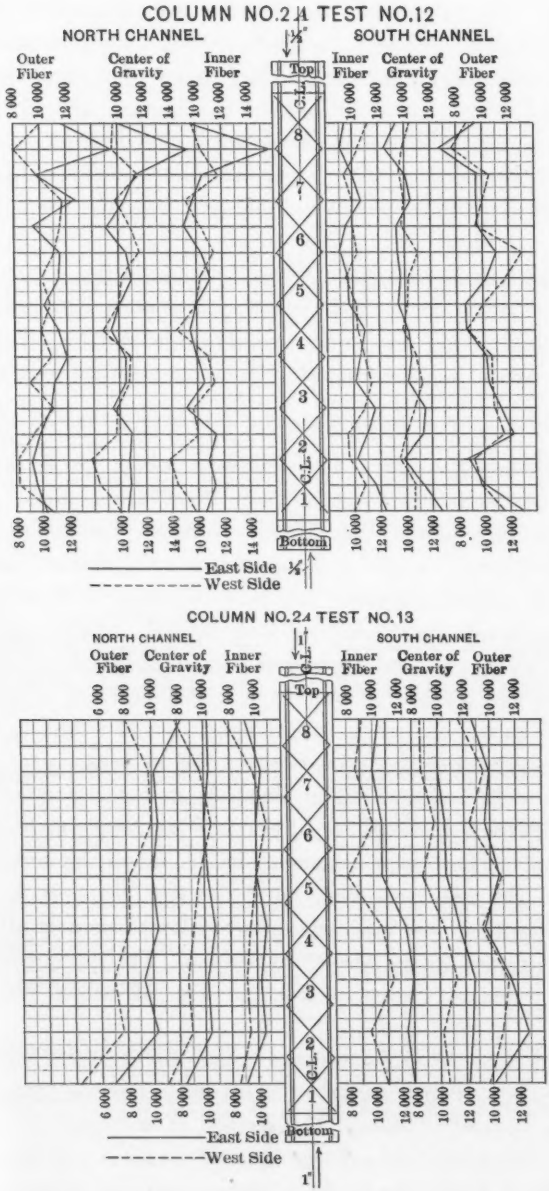
COLUMN NO.24 TEST NO.12
NORTH CHANNEL
SOUTH CHANNEL
Outer Fiber
Center of Gravity
Inner Fiber
Top
Bottom
East Side
West Side
COLUMN NO.24 TEST NO.13
NORTH CHANNEL
SOUTH CHANNEL
Outer Fiber
Center of Gravity
Inner Fiber
Top
Bottom
East Side
West Side

FIG. 10.

with Ames test gauges, and the actual curve assumed by the column under transverse load was thus determined. The theoretical elastic curve was computed from the common theory of flexure, not counting the lattice bars in the calculation of the moment of inertia. Fig. 2, Plate XI, shows the deflectometers and extensometers on Column No. 1 under the cross-bending test. Fig. 14 shows the deflection curves given by the column under transverse load and also the computed elastic curves.

It will be noted that when tested with the lacing vertical, Column No. 1 shows much greater deflection than that computed from the usual beam formula, while the stiffer wrought-iron column shows a much closer agreement with the curve, the heavy lacing apparently adding stiffness.

Field Tests of Columns.

Description of Bridge.—The bridge on which the field tests were made is an eight-panel single-track Pratt truss which spans the Sangamon River near White Heath, Illinois. The bridge is on the line of the Illinois Central Railroad between Champaign and Clinton, Illinois. The span measured 158 ft. 6 in. Fig. 12 gives a diagram of the bridge and Fig. 13 is from a photograph of the bridge.

Loads.—The test load applied to the bridge consisted of a mogul locomotive and tender followed by a loaded coal car and a caboose. Fig. 11 shows the test train, with dimensions and weights. This train was furnished through the courtesy of the railroad.

Members Investigated.—The members studied for stress distribution were Posts U_2L_2 South, U_3L_3 South, U_3L_3 North, and upper chord U_3U_4 South. The bridge diagram, Fig. 11, shows the location of the members. The posts were made up of two steel channels, double laced, while the top chord was made up of two built-up channels with a cover-plate on top and lacing across the bottom. The cross-section of the members tested is shown in Fig. 1. Fig. 12 shows the post and the chord.

Measurement of Deformation.—Ames test gauges were used as extensometers, and the method of attachment was the same as in the laboratory tests of columns. The method of reduction of instrument readings to stresses at the extreme fibers of members was also the same.

Routine of Tests.—As in the laboratory tests the stress distribution in the channels and the lattice bars was studied. The method of

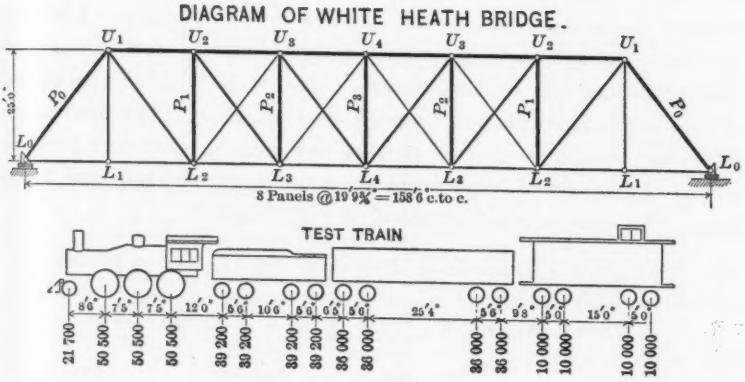
DIAGRAM OF WHITE HEATH BRIDGE.
U_1 U_2 U_3 U_4 U_3 U_2 U_1
P_0 P_1 P_2 P_3 P_2 P_1 P_0
L_0 L_1 L_2 L_3 L_4 L_3 L_2 L_1 L_0
25′0″
8 Panels @ 19′9¾″ = 158′6″ c. to c.
TEST TRAIN
8′6″ 7′5″ 7′5″ 12′0″ 5′6″ 10′6″ 5′6″ 6′5″ 5′6″ 25′4″ 5′6″ 9′8″ 5′0″ 15′0″ 5′0″
21 700 50 500 50 500 50 500 39 200 39 200 39 200 39 200 35 000 35 000 35 000 10 000 10 000 10 000 10 000

FIG. 11.

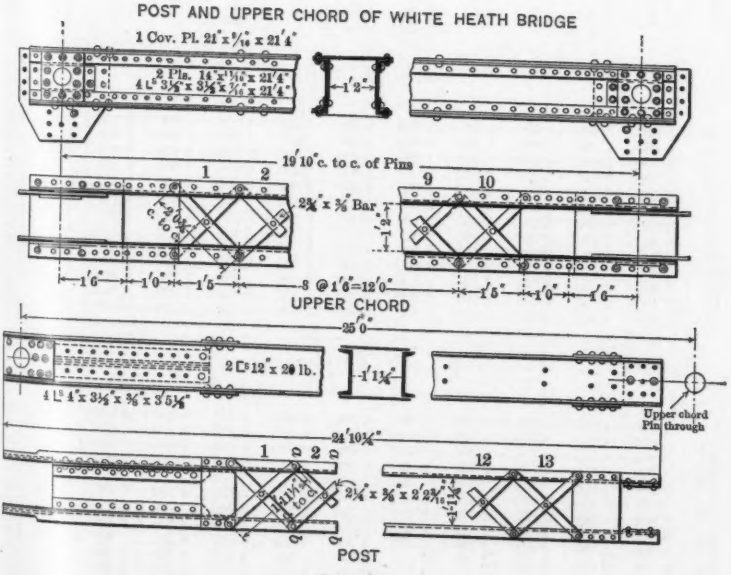
POST AND UPPER CHORD OF WHITE HEATH BRIDGE
1 Cov. Pl. 21″ x ⅝″ x 21′4″
2 Pls. 14″ x ⅜″ x 21′4″
4 L 3½″ x 3½″ x ⅜″ x 21′4″
1′2″
19′10″ c. to c. of Pins
1 2
9 10
2¾″ x ⅜″ Bar
1′2″
1′6″ 1′0″ 1′5″ 8 @ 1′6″ = 12′0″ 1′5″ 1′0″ 1′6″
UPPER CHORD
35′0″
2 L 12″ x 20 lb.
1′1¼″
4 L 4″ x 3½″ x ⅜″ x 3′5¼″
Upper chord Pin through
24′10¼″
1 2
12 13
2¼″ x ⅜″ x 2′2¾″
POST

FIG. 12.

procedure was as follows. Instruments were placed on some portion of the column to measure the deformation over a short gauge length, and a reading was taken. The test train was then run upon the bridge to a given position (one approximating the maximum load on the member under test), and the instruments were read again. The train was then run off the bridge, and the instruments were again read. This procedure was repeated several times, at least three applications of the load being made and frequently several more. The instruments were then moved to another part of the column, and that part was tested. Observations were made on both flange members and lattice bars. The tests covered a period of eight days.

TABLE 8.—STRESSES IN POSTS OF WHITE HEATH BRIDGE.

	North Channel.						South Channel.					
	East side.			West side.			East side.			West side.		
Panel.	Outer fiber.	Center of gravity.	Inner fiber.	Outer fiber.	Center of gravity.	Inner fiber.	Outer fiber.	Center of gravity.	Inner fiber.	Outer fiber.	Center of gravity.	Inner fiber.
Post U_3L_3 South.												
1*	5 000	5 300	5 400	2 600	2 700	2 700	4 900	5 000	5 000	2 300	2 500	2 600
2	3 400	3 500	3 500	1 900	1 600	1 500	3 300	3 400	3 400	1 600	1 800	1 800
3	3 500	3 800	4 000	1 800	1 900	2 000	2 600	2 900	3 100	2 600	2 200	2 100
4	3 700	3 400	3 300	2 800	3 000	3 000	1 600	3 100	3 600	2 400	2 800	2 900
5	3 500	3 900	4 000	2 900	2 900	2 900	2 800	3 000	3 100	2 200	2 600	2 800
6	3 600	3 600	3 600	3 400	3 800	3 900	3 400	3 400	3 400	4 100	3 400	3 200
7	2 600	3 200	3 400	3 300	3 100	3 000	2 400	3 000	3 300	2 600	2 700	2 800
8	3 300	3 200	3 200	3 900	3 600	3 500	3 600	3 400	3 300	3 700	3 900	3 900
9	3 200	3 100	3 000	3 600	3 300	3 200	1 700	2 500	2 800	2 900	3 300	3 400
10	2 800	2 700	2 700	3 100	3 300	3 300	2 700	2 400	2 300	3 300	3 700	3 900
11	3 300	2 900	2 800	4 500	3 400	3 100	2 900	2 000	1 700	3 500	3 900	4 100
12	1 400	2 000	2 300	3 100	3 200	3 200	1 800	1 800	1 800	4 800	4 700	4 000
13	1 300	1 500	1 500	2 800	2 700	2 600	2 600	2 900	3 100	5 200	4 900	4 800
Post U_3L_3 North.												
1	4 900	4 900	4 900	2 600	2 500	2 500	4 800	4 900	4 900	2 100	2 700	2 800
1½	4 800	4 700	4 700	2 900	2 800	2 700	3 100	4 900	5 700	900	1 500	1 700
2	4 800	4 400	4 300	2 600	2 500	2 500	4 900	4 700	4 600	2 700	1 700	1 300
2½	4 000	3 900	3 900	2 700	2 800	2 900	4 300	3 700	3 500	2 300	1 800	1 600
3	3 900	4 200	4 300	2 800	2 900	2 900	.....	.....	.....	.....	.....	.....
3½	3 900	3 900	4 000	2 400	2 500	2 500	3 200	3 800	4 100	2 400	2 500	2 500
4	3 400	4 600	5 000	2 700	2 700	2 700	3 900	3 700	3 700	3 500	3 200	3 100
4½	3 800	3 400	3 200	2 500	2 300	2 200	3 500	3 500	3 400	3 100	3 000	3 000
5	3 900	3 700	3 600	3 700	3 700	3 700	3 100	3 600	3 800	1 400	2 100	2 400
5½	4 100	3 600	3 400	4 000	3 400	3 200	3 500	3 900	4 100	3 500	2 900	2 800
6	3 600	3 200	3 100	3 400	3 300	3 200	3 700	3 500	3 400	3 900	3 600	3 500
6½	3 300	3 300	3 300	3 600	3 500	3 500	4 500	4 700	4 800	2 000	2 400	2 600

* No explanation for the high values in Panel 1 has been found. Five determinations of stress were made, including the removal and re-attachment of instruments.

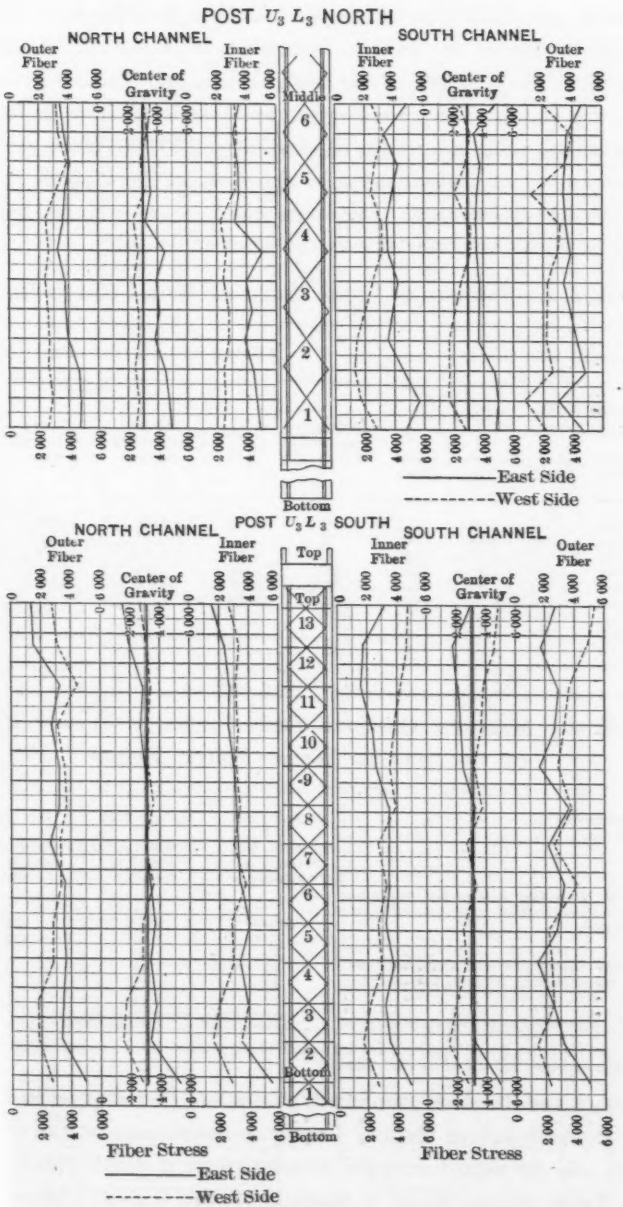
POST U_3 L_3 NORTH
NORTH CHANNEL
SOUTH CHANNEL
Outer Fiber
Inner Fiber
Center of Gravity
Middle
Bottom
East Side
West Side
POST U_3 L_3 SOUTH
Top
Fiber Stress

FIG. 13.

Results of Tests for Stress Distribution in Channels.—Table 8 gives the results of the tests to determine the stress distribution and variation in the channels of the bridge posts, and Table 9 gives those for the top chord. The stresses given are calculated from the observed deformations, using a modulus of elasticity of 30 000 000 lb. per sq. in. The conditions of measurement of deformation were much the same as in the laboratory tests. The stress noted is the average stress over a space of 4½ in. on either side of the point indicated.

TABLE 9.—STRESSES IN UPPER CHORD OF WHITE HEATH BRIDGE.

Lower Side (Laced).				Upper Side (Cover-plate).				
North Channel.		South Channel.		Distance from end, in inches.	North edge.	Over north web plate.	Over south web plate.	South edge.
Panel.	Outer fiber.	Inner fiber.	Outer fiber.	Inner fiber.				
1	4 300	3 900	5 000	5 500	33	5 500	6 000	5 200	5 700
2	4 900	4 300	4 500	4 800	57	.....	.....	5 200	5 500
3	2 800	5 500	3 400	4 500	81	5 200	5 400	5 900	5 300
4	2 800	5 100	4 200	5 300	105	5 000	6 100	5 700	6 000
5	5 700	5 400	3 900	4 800	129	4 800	5 900	5 900	5 900
6	5 500	4 800	5 100	4 200	153	5 400	5 700	5 500	5 600
7	5 900	4 700	3 400	3 600	177	5 000	5 000	5 700	5 600
8	6 200	5 300	3 300	4 400	.....	.....	.....	.....	.....
9	6 000	5 300	4 700	3 800	.....	.....	.....	.....	.....
10	5 900	5 100	4 400	4 900	.....	.....	.....	.....	.....

TABLE 10.—MAXIMUM OBSERVED FIBER STRESS IN FLANGE MEMBERS OF COLUMNS IN WHITE HEATH BRIDGE.

	U_3L_2 South F 1	U_3L_2 North F 2	U_3U_4 South F 3
Column number			
Test number			
Lacing	Double, 45°, riveted at crossing.	Double, 45°, riveted at crossing.	Cover-plate on top. Double, 45° on bottom.
Maximum observed compressive stress in an extreme fiber, in pounds per square inch	5 200	5 700	6 200
Percentage of excess maximum observed stress over average, highest five values.	73*	64	20
	63	48	19
	60	41	17
	57	31	17
	47	31	17

* No values from Panel 1 have been included in this table, as no explanation is known of the high stresses indicated in that panel.

Figs. 8 and 13 show graphically the stress distribution and variation. In these figures the full lines give the stresses at the west side (front), and the dotted lines the stresses at the east side (back).

In Table 10 are given a number of the highest observed fiber stresses. The excess of the maximum fiber stress is given as a per-

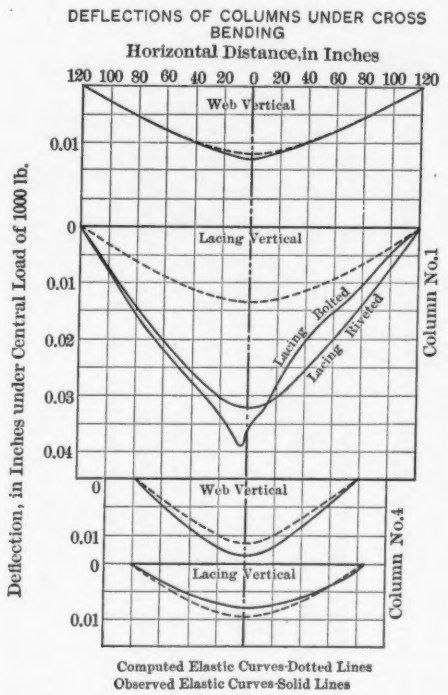

FIG. 14.

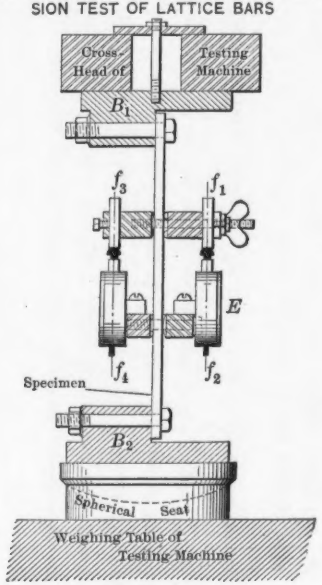

FIG. 15.

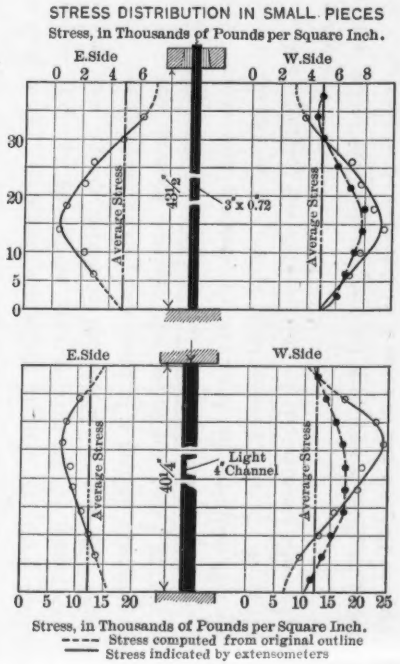

FIG. 16.

centage of the average stress. At most sections the maximum stress was in the outer fiber of the channel, but in some cases it was found at the inner fiber.

In the tests of the bridge posts an attempt was made to determine the stresses in a few of the lattice bars. These stresses were very small, and the precision of the extensometer was not sufficient to measure them with any great degree of accuracy. It should be noted that the lacing of the posts in this bridge was double, and the bars were riveted together at their intersection. In several cases it was found that a lattice bar under load bent in the shape of a very flat S curve, the point of attachment to another lacing bar, at the middle, being a point of inflection.

Special Tests on Bridge Columns.—Tests were made on the batten plates at the top of one of the posts, and under load a slight bending of the plates between channels was found. The bending took place in a horizontal plane.

In one post the change of stress was observed as the locomotive and train moved across the bridge. Extensometers were placed at *aa, bb* (Fig. 12), and on the floor beams, and the changes in reading were noted as the train moved across the bridge. In the inner channel of the post, tension was set up as the locomotive came opposite the post.

Tests of Lattice Bars, Small Columns and Column Material.

Compression Tests of Lattice Bars.—Many of the lattice bars in a column, as they transmit stress from one flange member of the column to the other, are under compression. To study the action of lattice bars under compression, a series of tests on single lattice bars was made. Fig. 15 shows the arrangement of the apparatus. The lattice bar was tightly bolted to the blocks, B_1 and B_2. The upper block, B_1, was fastened to the cross-head of a testing machine, and the block, B_2, was pressed against the weighing table of the testing machine. A spherical-seated bearing block was used, to insure an even bearing. Ames test gauges, E, mounted on suitable frames, were attached to the lattice bar over a short gauged length. From the readings of these gauges, the deformation of the extreme fiber of the bar was computed.

In this test of lattice bars, the load was applied with an eccentricity approaching that to be expected in a column for the lattice bars which

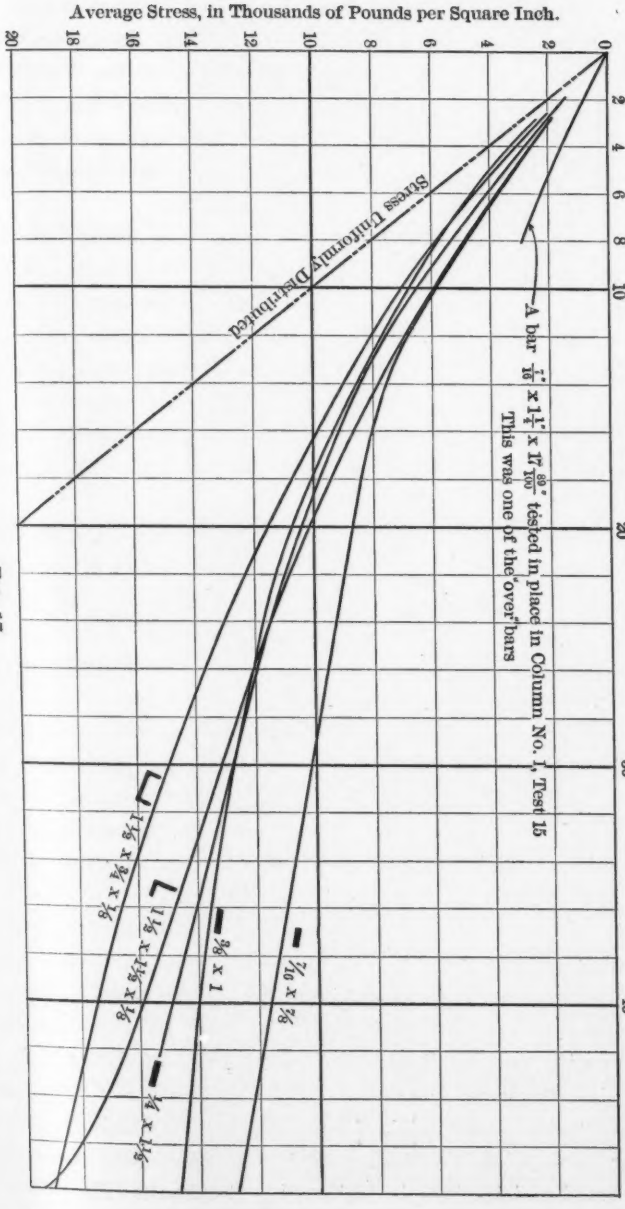
DIAGRAM OF FIBER STRESSES IN COMPRESSION TESTS OF LATTICE BARS.
Average Stress, in Thousands of Pounds per Square Inch.
Maximum Stress, in Thousands of Pounds per Square Inch.
Stress Uniformly Distributed
A bar 1/16" x 1 1/4" x 17 89/100" tested in place in Column No. 1, Test 15
This was one of the 'over' bars
1/2 x 1/2
1 x 3/8
1 1/8 x 1 1/8 x 3/8
1 1/8 x 3/4 x 3/8
1/4 x 1 1/8

FIG. 17.

are next to the flange member (here designated "under" bars). The lattice bars outside of these "under" bars are here designated "over" bars. The stress distribution across the section of the "over" bars, which are under compression, is probably more uneven than the stress distribution found in these tests. However, these tests give some idea of the relative behavior of lattice bars of various proportions, and of the large eccentricity of loading of all lattice bars.

Lattice bars of the following cross-sections were tested: Flat bars $1\frac{1}{2}$ by $\frac{1}{4}$ in., 1 by $\frac{3}{8}$ in., $\frac{7}{8}$ by $\frac{7}{16}$ in.; angles $1\frac{1}{2}$ by $1\frac{1}{2}$ by $\frac{1}{8}$ in.; channels $1\frac{1}{2}$ by $\frac{3}{4}$ by $\frac{1}{8}$ in. Several channel and angle lattice bars were tested with ends flattened and ribs turned inward, to minimize the eccentricity of loading. Bars of the following lengths between centers of rivet holes were tested: $8\frac{1}{2}$ in., $13\frac{1}{2}$ in., and 20 in. The rivet holes were $\frac{1}{2}$ in. in diameter. All bars were tested in a Philadelphia Machine Tool Company's 100 000-lb. testing machine, and loads and extensometer readings were taken to failure.

Observations were also made on the behavior of a lattice bar in a column under load, with a view to determine the distribution of stress over the section. Column No. 1 was loaded obliquely. The instruments were placed on an "over" bar which had been found to carry a high compressive stress, and readings were taken to determine the distribution of stress across the section.

When Column No. 1 was under cross-bending test, observations were made to determine the stresses transmitted by lattice bars and their distribution over the section of the bars. Extensometers were placed successively on most bars under compression on one-half of the column, and on some bars which were under tension. In both of these tests the bars were $1\frac{1}{4}$ by $\frac{7}{16}$ in., and were riveted.

Results of Tests of Lattice Bars.—The results of the tests of single lattice bars are given in Figs. 17 and 18, and in Tables 11 and 12. Fig. 17 shows the ratio of maximum to average stress in the bars $13\frac{1}{2}$ in. long between centers of rivet holes. It also gives the result of the test of stress distribution in a lattice bar of Column No. 1. Table 11 gives the stresses at failure of the various bars tested singly. The average stress on the various bars, which corresponds to a maximum fiber stress of 40 000 lb. per sq. in., as taken from these tests, has been noted and is given in Table 12. The results of the tests to failure are shown graphically in Fig. 18. The angle and channel bars

tested with flattened ends and ribs turned inward failed in the flattened part at loads no greater than the bars fastened with ribs turned outward.

TABLE 11.—COMPRESSION TESTS OF LATTICE BARS.

Average of two specimens.

Section of bar, in inches.	Distance from center to center of rivet holes, in inches.	$\frac{l}{r}$	Average stress at failure, in pounds per square inch.	Stress at failure for steel of 40 000 lb. per sq. in. yield point, in pounds per square inch.*
1½ by ¼, flat	20	277	9 900	8 900
1 by ⅜, flat	20	184	12 900	12 200
⅞ by 7/16, flat	20	158	14 500	15 000
1½ by ¾ by ¼, channel	20	90	20 800	19 400
1½ by 1½ by ⅛, angle	20	43	22 600	20 100
1½ by ¼, flat	13½	187	15 400	13 800
⅞ by 7/16, flat	13½	107	16 300	16 800
1 by ⅜, flat	13½	124	16 900	15 900
1½ by ¾ by ¼, channel	13½	61	Bolt sheared.	.....
1½ by ½ by ¼, angle	13½	29	Bolt sheared.	.....
1½ by ¼, flat	8½	118	17 300	15 500
⅞ by 7/16, flat	8½	67	18 100	18 700
1 by ⅜, flat	8½	78	18 300	17 200
1½ by ¾ by ¼, channel	8½	38	Bolt sheared.	.....
1½ by 1½ by ⅛, angle	8½	18	Bolt sheared.	.....

*The values in this column were obtained by multiplying the observed stress at failure by $\frac{40\,000}{\text{yield point determined from tests.}}$

TABLE 12.—AVERAGE STRESS IN LATTICE BARS WHICH CORRESPONDS TO A MAXIMUM FIBER STRESS OF 40 000 LB. PER SQ. IN.

Section of bar, in inches.	Distance from center to center of rivet holes, in inches.	Corresponding average stress, in pounds per square inch.
⅞ by 7/16, flat	13½	11 600
1 by ⅜, flat	13½	14 000
1½ by ¼, flat	13½	14 900
1½ by 1½ by ⅛, angle	13½	15 900
1½ by ¾ by ¼, channel	13½	17 500

Table 13 gives the results of the test for stress distribution in the lattice bars of Column No. 1 as it was stressed in cross-bending.

Tests of Small Columns.—Tests of two small compression pieces were made in order to study the effect of slight bends and kinks in the column upon the distribution of stress. The deviation from a straight line, in these nominally straight pieces, was measured before the load was applied. The deformations on two opposite faces for a given load were measured. The extensometer was similar to that used on the

single lattice bar tests. The instrument was shifted from one position to another along the column. The columns were finally loaded to failure. One of the columns was a flat piece of steel, 3 by 0.72 in. in cross-section, and 40 in. long. It was held at the upper end by wedge grips in the cross-head of the machine and at its lower end rested on a spherical-seated block. The second compression piece was a 4-in. channel 40 in. long. The ends were planed square; the upper end bore on a flat compression plate in the iron head of the machine, and the lower end rested on a spherical-seated block.

TABLE 13.—Stress in Lattice Bars of Column No. 1 under Cross Bending.

Column tested as a beam centrally loaded over span of 19 ft. 8 in.; lattice bars $1\frac{1}{4}$ by $\frac{7}{16}$ in.; 17.89 in. center to center of rivet holes; rivets $\frac{1}{2}$ in. in diameter.

Bar.	Maximum fiber stress, from Ames dials.	Average stress, from Ewing extensometer.	Ratio, maximum to average.
	"Over" Bars in Compression.		
14*E*	6 000	2 600	2.31
16*E*	8 300	2 000	4.15
18*E*	4 300	1 800	2.39
20*E*	7 700	1 400	5.50
22*E*	7 000	2 900	2.42
24*E*	8 900	2 200	4.04
			Average, 3.47
	"Under" Bars.		
15*W*	4 300*c*	2 500*c*	1.72
25*E*	3 800*t*	3 100*t*	1.23
17*E*	3 900*t*	3 000*t*	1.30
25*W*	3 900*c*	2 800*c*	1.39
19*W*	4 500*c*	2 900*c*	1.55
			Average, 1.44
	"Over" Bars in Tension.		
16*W*	7 000	3 000	2.33
18*W*	7 000	2 200	3.19
20*W*	4 500	2 400	1.87
22*W*	5 700	2 200	2.59
24*W*	7 000	2 700	2.59
26*W*	3 800	2 200	1.73
			Average, 2.38

Fig. 16 gives the results of tests of the small columns. The dotted line represents the maximum fiber stress computed by considering the eccentricity of loading at any cross-section to be equal to the deviation of that section from a straight line connecting the ends of the column. The deflections were slight, and were neglected in the calculations. The solid line represents the stresses on the two sides of the column, as determined from the extensometer readings.

TABLE 14.—TENSION TESTS OF MATERIAL FROM COLUMNS.

Test piece from:	Material.	Number of test pieces.	Average stress at yield point, in pounds per square inch.	Average stress at ultimate, in pounds per square inch.	Average Elongation. Percentage
Column No. 1 Angles.....	Steel.	2	43 300	61 600	37
$\frac{7}{16}$ by 1¼-in. lacing bar.....	Steel.	2	40 700	58 100	42
1 by ½-in. lacing bar.....	Steel.	1	42 400	62 000	38
Channels of wrought-iron columns.	Wrought iron.	3	30 700	46 800	17

Tests of Column Material.—Table 14 gives the results of the tensile tests of samples of material from various parts of the flange members of the columns, and also of tension tests of lattice bars. Table 15 gives tension tests of lattice bars like those used in the compression tests of lattice bars.

TABLE 15.—TENSION TESTS OF LATTICE BARS.

Shape.	Dimensions of cross-section of whole bar, in inches.	Number of specimens tested.	Average stress at yield point, in pounds per square inch.	Average stress at ultimate, in pounds per square inch.	Percentage of elongation in 2 in.
Channel.	1½ by ⅜ by ¼	1	43 000	57 600	36.5
Angle....	1½ by 1½ by ⅛	2	45 000	59 300	24.5
Flat.....	⅞ by $\frac{7}{16}$	2	38 700	57 000	45.5
Flat.....	1 by ⅜	2	42 400	61 700	45.8
Flat.....	1½ by ¼	2	44 600	60 800	42.8

COMPARISON AND DISCUSSION.

The Action of Built-up Compression Pieces.—In analytical discussions of column action, the stress is usually assumed to vary uniformly from a minimum on one side of the cross-section to a maximum on the opposite side, and the whole cross-section of the column is considered to act as a unit. The longitudinal axis of the column is also considered to take a definite elastic curve under load. In the

derivation of most column formulas, it is assumed that the amount of deflection of the elastic curve from the original position of the axis is an important element in fixing the maximum stress in the column. Although these assumptions are generally used as the basis of column formulas, it may be well to consider whether conditions may not exist, in columns of ordinary form and dimensions, which will render doubtful the general applicability of some of these assumptions, and will dwarf the effect of others. At any rate, it seems worth while to consider the effect of other conditions in a built-up member. It must be borne in mind that the built-up column is subject to imperfections of fabrication, and that some crookedness and eccentricity must exist.

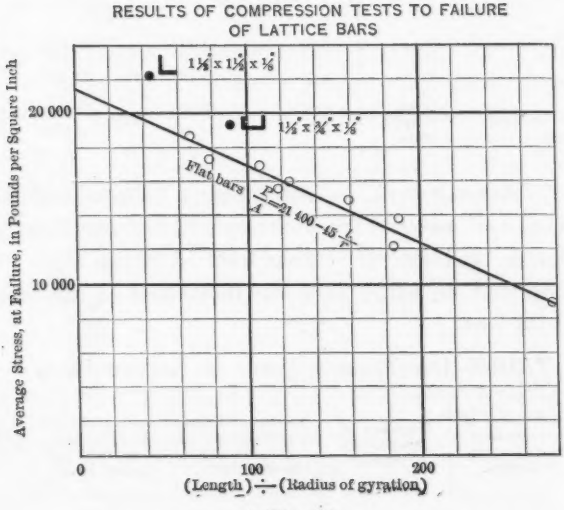

FIG. 18.

The component parts of the column may be relatively slender and flimsy. Whether there is integrity of cross-section under load, is a question. In the tests herein described, the amount of deflection from the original axis, for loads up to a point somewhat below incipient failure, was found to be slight (generally between 0.04 and 0.1 in.), much smaller than necessary to account for the stresses observed in the columns.

The action of short columns at failure may be expected to be different from that of longer columns, although the stresses up to incipient failure may be the same. Granting that the conditions of

non-straightness are such that the distribution of stress over the cross-section is the same for the two lengths of column, and that the deflection of the column is so slight as not to affect materially the stresses developed, the longer column will be in more danger of immediate and sudden collapse after the yield point of the material in any fiber has been reached, and the total load carried before complete failure will, in general, be less. This is because, in a ductile material, after the stress at one side of the column has passed the yield point, the total resistance of the section to compression will increase, while the resistance to cross-bending may not. Under the conditions named, the bending moment due to eccentricity will be the same until the yield point in some fiber is reached. After yielding begins, the greater deflection in the longer column rapidly increases its relative eccentricity, and more rapid failure may be expected than with the shorter column.

Indications of Data.—It will aid in the interpretation of the data of the distribution of stress over the channel members of the columns to point out a few simple indications. Reference may be made to the diagrams in Figs. 6, 7, 8, and 13, and to Tables 2, 3, 8, and 9.

1.—Any lack of agreement between the average load stress and the average of the stresses given for the four centers of gravity of channel flanges may be ascribed to errors of observation.

2.—If the stress at the center of gravity of one channel is above the average stress throughout the length of the column, and the corresponding stress for the other channel is similarly below the average stress, there must be an eccentricity in the application of the load at the two ends. If the stresses at the center of gravity of one channel member forms in the diagram a straight line which crosses the line of average stress, and that for the other channel crosses in the opposite way, the eccentricity of the load application must be oblique.

3.—If the stress at the center of gravity of a channel in near-by points is greater first in one channel and then in the other, the change may be due to crookedness of the column throughout that part of the length.

4.—If, in one channel or in one channel flange, the stress at the center of gravity remains constant and that of the extreme fiber varies, the change may be due to local crookedness of this channel and there will be a lateral bending of this member.

5.—If the front side of a channel has a higher stress than the back side, there must be bending action through its web, and *vice versa.*

6.—Changing stresses in the diagonally opposite corners of a channel may indicate twisting of the channel, and another combination of stresses may indicate a twisting or oblique distortion of the column as a whole.

An inspection of the diagrams shows that all these indications are found in the tests.

Does the Built-up Column Act as a Unit?—Engineers have often expressed doubt as to whether the parts of a built-up column act as a unit, although column formulas assume this unity of action. The tests throw some light on the question of the integrity of cross-section under load. The individual channel, of course, acts as a unit to resist bending action, though there are indications of twisting. The integrity of the whole section with reference to a plane parallel to the lacing seems probable, except as twisting action exists. With reference to a plane through the axis perpendicular to the plane of the lacing, this unity of action is not so certain. The tests on the distribution of compressive stress and likewise the cross-bending tests indicate that these built-up columns did not in all cases act as a unit but rather as two members not fully restrained by the lacing. The stresses in two channels at points in the same cross-section do not give the regularity of variation which would exist if the column bent as a unit. The elastic curve assumed by Column No. 1 under cross-bending load, shown in Fig. 2, Plate XI, differs from the computed elastic curve, though that for the wrought-iron column gives little difference. In the case of the posts of the White Heath Bridge, however, there is much closer agreement and a seemingly closer approach to unity of action.

Effect of Non-straightness of Built-up Columns Upon Distribution of Compressive Stress.—The effect of crookedness or other irregularities of a constituent member of a built-up column may be realized if a rough analysis of the case be made. Consider a part of one of the channels forming a column, taking the length between the connections of two adjacent lattice bars. This member is under compression. Owing to non-straightness or to the non-homogeneity of the material, the load on this short piece is not evenly distributed over the section;

that is, it is not centrally loaded, but may be considered to have an eccentricity with respect to the gravity axis. Call this eccentricity e (Fig. 19). Neglect any deflection of the piece under consideration due to the load. Call the compression load coming on this piece P; A its area of cross-section; I its moment of inertia about YY, and r the corresponding radius of gyration; and c the distance from YY

ECCENTRICITY IN CHANNEL MEMBER

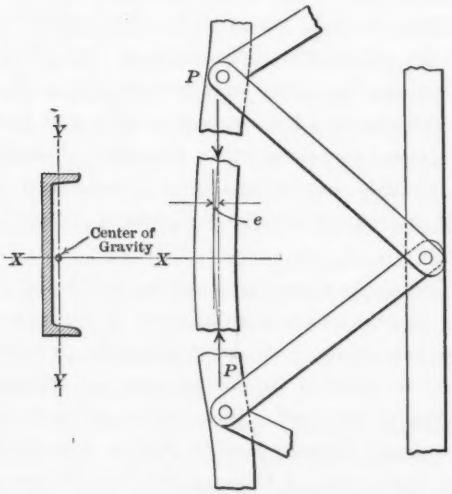

FIG. 19.

to the remotest fiber. Then the bending moment due to the eccentricity is Pe. The maximum stress will be

$$f = \frac{P}{A} + Pe\,\frac{c}{I} = \frac{P}{A}\left(1 + \frac{e\,c}{r^2}\right).$$

The excess of the stress in the extreme fiber of the piece over the average stress, produced by the eccentricity, e, is then $\frac{P}{A}\,\frac{ec}{r^2}$, and hence the term, $\frac{ec}{r^2}$, gives the proportionate excess of stress in the extreme fiber. This value is applicable to the channel, or to one flange of the channel, or it may be applied to the column as a whole by using the properties of the whole section. In the single channel under consideration, c is relatively large and r is relatively small, and the excess of maximum stress for a given eccentricity, e, may be expected

to be relatively large. It will be seen that for an excess of 50% in the extreme fiber of a channel of Column No. 1, *e*, by this formula, would be 0.045 in., and, in the wrought-iron columns, 0.057 in. A slight variation from straightness in a channel will account for considerable increase of stress.

Excess of Maximum Fiber Stress over Average Stress in Channel Members.—The diagrams and data show that the compressive stress is unevenly distributed over the cross-section of the columns tested, and also that there is great variation in this distribution at the various sections along the length of the column. It will be noted that in a number of sections the excess of stress was from 40 to 50 per cent. In one test of Column No. 1, an excess of 67% was found, and in the White Heath Bridge an excess of 73 per cent. Possibly these values were unusual, or the observations were erratic, but the indications of a fiber stress of from 40 to 50% in excess of the average stress were not uncommon.

It may be seen that among the causes to which the high fiber stress may be attributed are (*a*) non-straightness of the column as a whole, (*b*) non-straightness of the component channels, or eccentricity in the delivery of stress to them by the lacing, and (*c*) unknown eccentricity in the application of the load. It would be of interest to know how much of the increase of stress may be due to any one of these conditions. A study of the tests of Column No. 1 shows that generally only a small amount may be said to be due to non-straightness of the column as a whole. In but few cases is it found to be more than, say, 5 per cent. In four places it seems that the excess attributable to this may be estimated to be between 20 and 25 per cent. The effect of non-straightness of the individual channels seems to be greater. At several points the excess of stress attributable to this cause appears to be from 30 to 50 per cent. As already noted, a kink in the channels of 0.045 in. would give, by the analysis made, an eccentricity sufficient for a 50% increase in stress. Not all of this crookedness need be between adjacent rivet points, as the stress may not reach normal for some distance on either side. The effect of the third condition, eccentricity of application of the load, will vary with the construction. In Column No. 1 the effect of undetermined eccentricity of application of load appears to be not nearly as great as the effect of non-straightness of the component channels.

In the wrought-iron columns, which are much stockier, the lack of straightness in individual channels has less effect, seemingly less than 15%, and much the larger part of the high fiber stresses appears to be due to general column eccentricity or to eccentricity of loading.

The results for the posts of the White Heath Bridge are of interest in this respect. It is evident that the effect of non-straightness of channels was not very large, and also that effect of non-straightness in the column as a whole was relatively small. There is, however, an evident bending in the direction of the web of the channels. For example, in U_3L_3 South, the back side of the channels has the maximum stress at the top and the front side at the bottom. The bending moment producing this may be due to obliquity of end pressures or to a bending by the connecting floor-beam and top chord. A twisting action is also apparent. Post U_3L_3 North gave quite similar results.

Compressive Strength of Lattice Bars.—In the discussion of stress developed in column lacing, the stress considered was the average over the bar. As usually attached, there is considerable flexure in the bar, and the ability of the bar to carry this eccentric load should be considered. The bars are most likely to fail in compression, since they act as long columns eccentrically loaded.

The tests of individual lattice bars (Fig. 17 and Table 12) show that the maximum fiber stress may be several times the average stress. It is also seen that even in a short lacing bar the maximum load carried is only about one-half the yield point of the material. The necessity of using very low working loads on lattice bars appears to be important. It will be noted that at low stresses there is similarity of distribution of stress in the slender bars and in the thicker bars, but the slender bars fail at smaller computed fiber stress.

The results of tests to destruction of individual lattice bars are fairly well represented by the formula:

$$\frac{P}{A} = 21\,400 - 45\frac{l}{r}$$

where P = load at failure, in pounds, A = area of cross-section, in square inches, l is the distance, in inches, from center to center of rivet holes, and r is the radius of gyration, in inches, of the cross-section of the lattice bar. These results may be considered to be applicable to "under" lattice bars. For "over" bars it seems probable that the average stress at failure would be considerably less.

Effect of Cover-Plates and End Connections.—In the tests of the White Heath Bridge, the effect of the cover-plate seems striking. The upper chord, U_3U_4, composed of two built-up channels with one cover-plate, gave an excess fiber stress of 20% at the worst section, while the posts, composed of two channels laced on both sides, gave a maximum of 73 per cent. The high value in the posts may be due to other causes, but it seems reasonable to expect that the cover-plate will act to reduce the irregularities in fabrication.

The connections of the ends of the posts evidently exerted a very noticeable effect on the stress distribution. In one of the posts tested, the stress was greatest at one corner of the post at the top and at the diagonally opposite corner at the bottom. It will be remembered that the posts were riveted to the top chord, and were connected with the lower chords by pins. The floor-beams were riveted to the sides of the posts, and this connection affects the stress distribution. Readings of deformations taken on the floor-beams and posts show that the loaded beam was partly restrained at the ends by the post and that there was an appreciable bending in the post.

Stresses in Column Lacing.—If the load carried by one channel of a column were the same throughout its length, no stress would be carried by the lattice bars. Such stress is developed whenever there is a change in the relative amount of loads carried by the two channels. If at the section, *AB* (Fig. 2), there is an equal division of load between the two channels, and also at the section, *CD*, and if at some section, *EF*, the division of load is unequal, it is evident that the lattice bars must be called into action to transmit this stress, and that transverse shear exists in the interval. In general, the conditions producing this must be complex, rendering analysis unsatisfactory, except in so far as the shear may be due to a known eccentricity of loading.

It is evident from the tests that the relative stress in the two channel members varies considerably from end to end and that the stress in lattice bars also varies. It seems probable that the transverse shear developed may be traced largely to irregularities in outline, or at least that these irregularities may be expected to cover up other causes of stress in the lacing of centrally-loaded columns, if we include in such irregularities all unknown eccentricity.

The amount of transverse shear necessary to produce the maximum

observed lattice-bar stress (given in Table 6) is of interest, though of course it cannot be taken to be conclusive. The measurements were generally made at working loads. So far as observations were made on columns tested to failure, the distribution of stress remained much the same up to incipient failure. The values given in Table 6 indicate maximum average stresses in the bars such as would be caused by a transverse load ranging from 2 to 6% of the central compression load or of a transverse shear one-half as great.

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PAPERS AND DISCUSSIONS

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CAISSON DISEASE AND ITS PREVENTION.

By Henry Japp, M. Am. Soc. C. E.

To be Presented June 2d, 1909.

The prevention of caisson disease has not received as much attention as its cure, the general opinion being that it cannot be prevented. In forming an idea as to the possibility of its prevention, something may be learned from Nature's method of carrying out work by the aid of living organisms under air pressure.

First let us inquire whether Nature has a sufficient range of air pressure to make it necessary for a capacity to sustain variations without injury. The highest altitude on this earth is Mount Everest, 29 000 ft. above the level of the sea, and the deepest mine is Tamarack No. 3 Shaft, of the Calumet and Hecla Mines, 4 407 ft. below sea level.

The curve on Fig. 1 shows the absolute air pressures at various altitudes between these extremes. The total difference of 12½ lb. per sq. in., or from 9¾ lb. below atmospheric pressure at sea level (viz., 14.7 lb.) up to 2¾ lb. above normal, is considerable.

Mount Everest has never been ascended, but an elevation of 20 000 ft. has been reached, and men can live in comfort at an altitude of 7 000 ft., as in Mexico City, which has an air pressure of 3½ lb.

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

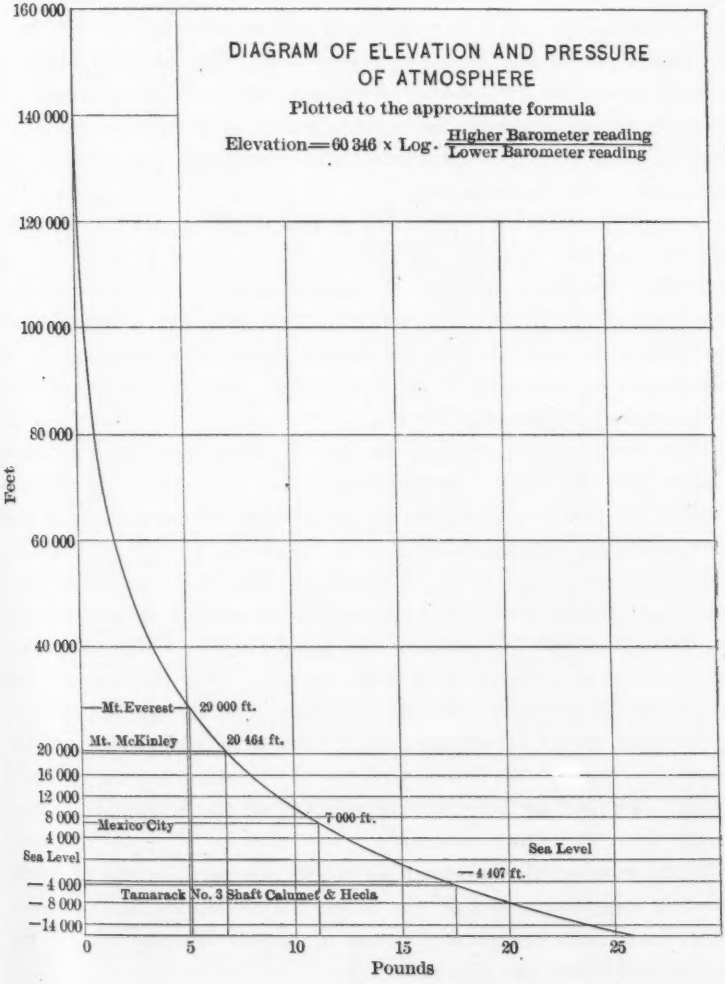
DIAGRAM OF ELEVATION AND PRESSURE OF ATMOSPHERE
Plotted to the approximate formula
Elevation = 60 346 x Log. Higher Barometer reading / Lower Barometer reading
160 000
140 000
120 000
100 000
80 000
60 000
40 000
20 000
16 000
12 000
8 000
4 000
Sea Level
— 4 000
— 8 000
—14 000
Feet
Mt. Everest
29 000 ft.
Mt. McKinley
20 464 ft.
Mexico City
7 000 ft.
Sea Level
— 4 407 ft.
Tamarack No. 3 Shaft Calumet & Hecla
0
5
10
15
20
25
Pounds

FIG. 1.

per sq. in. less than at the coast. Such a change of pressure is quite noticeable to those whose Eustachian tubes are closed, and, although the range of pressure is insufficient to produce caisson disease, yet travelers have complained of discomfort, not unlike a very mild attack, after a rapid journey by rail from Vera Cruz to Mexico City.

Eagles are known to go to great heights beyond the range of human vision, fish are capable of going to great depths in water, and beasts of prey roam from the mountain side to the valleys far below. With each descent these creatures have undergone compression, and with each ascent decompression.

It will be noted that Nature has no time limit for immersion under increased pressure, and the application of pressure is a much speedier and easier operation than that of decompression; and, while the compression is down-hill and attained with little effort, the decompression can only be attained by the continuous effort of climbing, and, in long ascents, with frequent rests or by stages. This is surely a forecast of the stage-decompression theory.

From Nature then, our best teacher, we learn that the time taken to enter the air-chamber may be short, the time spent in the air-chamber may be long, and the rate of coming out must be slow and accompanied by exercise.

In early compressed-air work engineers were confronted with caisson disease, and have long been striving for a remedy. Men of observation noted first that lock tenders who made frequent entry to the air-lock for short visits were much more immune from the malady than workers who stayed for a full shift, and from this argued that shortening the time in the air-chamber would be a safeguard, and, as higher pressures were used, the hours continued to be shortened until it was thought that half an hour at a maximum of 50 lb. gauge pressure was the limit of human endurance.

As the number of compressed-air works increased, so the workmen gained in experience, and it was noted that workmen of experience who suffered from the painful forms of caisson disease relieved the pain by re-entering the air-chamber.

About thirty years ago Paul Bert advanced the theory that caisson disease was caused by the nitrogen of the air being dissolved in the blood. He advocated slow decompression, suggesting 30 min. for 2 to 3 atmospheres (15 to 30 lb. gauge pressure) and 60 min. for 3 to 4

MEDICAL AIR LOCK USED IN THE EAST RIVER TUNNELS OF THE PENNSYLVANIA TUNNEL AND TERMINAL RAILROAD

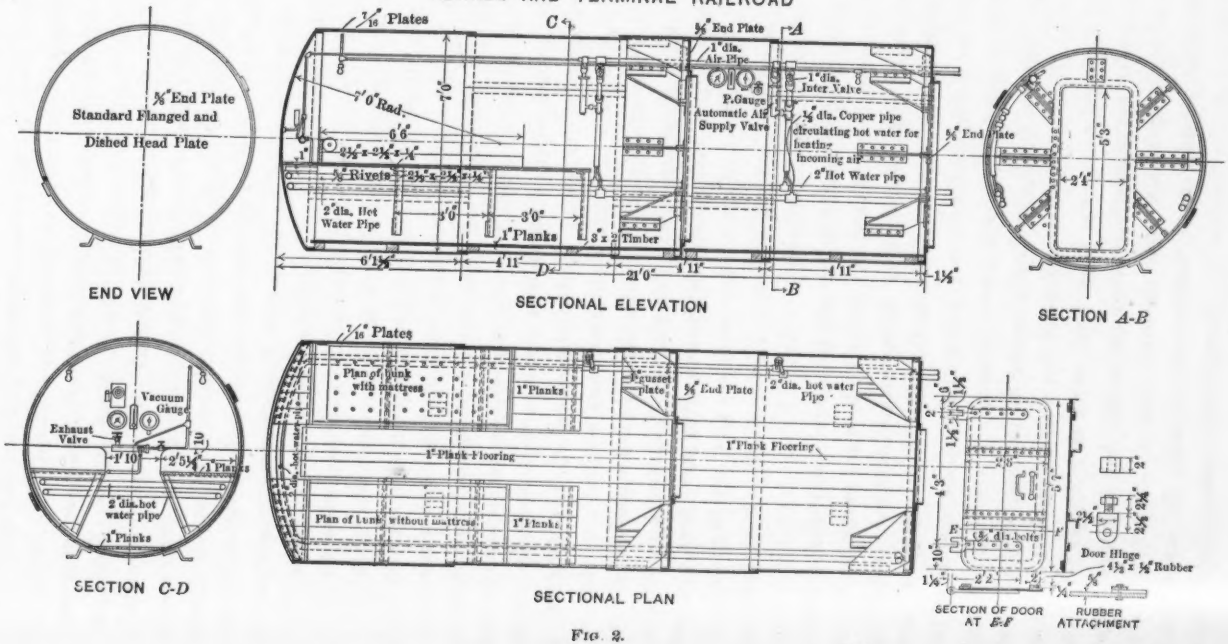

FIG. 2.

atmospheres (30 to 45 lb. gauge pressure). Since his time many writers and experimenters have followed in his footsteps, but it was not until Dr. Smith, of New York, suggested that a recompression chamber would act as a remedy, and E. W. Moir, M. Am. Soc. C. E., of London, without knowledge of Dr. Smith's prior suggestion, actually built a medical air-lock, or recompression chamber, in 1890, on the old Hudson Tunnel, New York, that any reliable cure for the trouble was discovered. Fig. 2 shows the medical air-lock designed by Mr. Moir, of which six were used on the East River tunnels of the Pennsylvania Tunnel and Terminal Railroad.

It was then found that if a man suffering from caisson disease in any of its many forms, varying from unconsciousness and paralysis to an acute pain in the limbs, was quickly recompressed and allowed to decompress slowly, in many cases a cure resulted, but not always.

This went to prove that the disease was a mechanical one, and was caused by the dissolved air in the blood and tissues becoming liberated in the body in the form of expanding bubbles which tore the tissues, caused pressure on the brain, injured the spinal cord, or frothed up the blood and stopped the circulation and heart action.

Much has been done since the introduction of the medical air-lock. The percentage of carbon dioxide in the air of the working chamber has been kept within safe limits, moisture and oil have been extracted from the air before delivery to the chamber, and the workmen have been well cared for; but still caisson disease claims its victims, and engineers consider this inevitable.

The erratic manner of the disease puzzles doctors and engineers alike. Men who have worked for months in high pressures for the regulation shift without suffering have been suddenly attacked and died; others, working only half a shift, have been paralyzed; while exceptions have worked for 12 hours at a time and have suffered only slight pains which have quickly passed away.

The writer, as Managing Engineer for S. Pearson and Son, Inc., on the East River Tunnels contract for the Pennsylvania Tunnel and Terminal Railroad Company, New York, with as many as ten tunnel headings under compressed air simultaneously, has had many opportunities of studying this strange disease.

When these tunnels were commenced, in 1904, the knowledge of compressed air at the disposal of the contractor was quite extensive,

and included the long experience of Alfred Noble, Past-President, Am. Soc. C. E., on caisson work in the United States, and Mr. E. W. Moir's experience on the Forth Bridge caissons, the old Hudson Tunnel, and the Blackwall caissons and tunnel.

In the light of past experience, the general conduct of the work was framed on a few established rules which when condensed amount to the following: No workman was allowed to enter the air-chamber without a physical examination by the qualified medical officer of the contractors. Sound physique was the chief requirement. The men were cautioned not to enter the air on an empty stomach, to wear warm clothing on coming out, and to drink hot coffee.

The time worked in the air-chamber was limited to 8 hours with half an hour off for lunch, up to 32 lb. gauge pressure, and two spells of 3 hours each with 3 hours rest between for pressures from 32 to 42 lb., and two spells of 2 hours each for pressures greater than 42 lb. with 4 hours rest between, with no limitation as to decompression. Two medical air-locks were installed on each side of the river, well-warmed dressing rooms were provided for the workmen, and there were covered gangways for access to the shafts.

The air was cooled before delivery to the tunnels, and samples were taken in the tunnels by Mr. Noble's engineers and analyzed daily. The air was regulated so that the carbon dioxide did not exceed 10 parts in 10 000, and the tunnels were kept in a sanitary condition.

Owing to the grade of the tunnels being so deep on the Manhattan side of the river, the air pressure very quickly rose to 36 lb.

Practically no cases of bends occurred until the pressure reached 29 lb., and then, within a few days of each other, two men died. These men had entered the air-chamber without being passed by the doctor. Then it became necessary to post outside each air-chamber a guard whose duty it was to keep out of the tunnel men who had no doctor's pass. At this time, reliance could not be placed on the tunnel foremen, as they were likely to be absent from work, and new men had to be selected each day.

For many months after the work started, while the men were being seasoned, the tunnel gangs and foremen were in a state of change, owing to the difficulty of getting good men and the frequent absences due to caisson disease, and it was a long time before an efficient organization was built up.

As the tunnels were driven deeper beneath the East River the pressure quickly rose, and ultimately reached 36 lb., gauge pressure, with only one set of air-locks in operation; but even the change at 32 lb. from one shift of 8 hours to two shifts of 3 hours each gave no relief, and cases of bends, sometimes fatal, continued all the time.

It was not long after 27 lb. was reached that the more sensitive members of the staff found that it did not pay to come out quickly, and at 30 lb. pressure it became a custom to take about ½ min. for each pound. After one or two additional fatal cases occurred, it was decided to limit the workmen to approximately the same rate of decompression, or actually 15 min. for 35 lb. pressure.

Many of the men complained that taking so long to decompress gave them caisson disease, and it was difficult to compel them to take long enough. The guards at the entrance to the tunnels had now to record the time taken to decompress, and, as the workmen frequently used the lower muck locks as well as the upper man lock, it was impossible to tell when decompression commenced owing to the noise of exhausting air. Therefore it became necessary to run a small ¼-in. pipe from the exhaust pipe of each lock to the cabin in which the guard was stationed, a whistle was attached, and a small ball of cotton was suspended by a light string over each pipe. The clerk, noting when the ball was puffed off for each lock, booked the workmen off as they left the lock. The material locks were fitted with inner material valves and, in addition, man valves of smaller size, and a pressure gauge and a clock were fixed in each air-lock.

As the guard had already booked the men as they entered, he could tell if any were exceeding the regular working shift, and his record was valuable for checking the time-keepers. This rough method of checking the duration of the decompression was quite good enough for the purpose. An attempt to improve it was made when a 12-in. Crosby recording gauge was installed on one air-lock, but it was ineffectual because the air-lock was often sent out or decompressed with no one inside, and this complicated the record, which was much too small, and involved considerable trouble in locating the record of decompression for individual gangs. No doubt a suitable recording instrument could be devised for this special purpose.

The effect of lengthening the decompression period to 15 min. reduced the number of cases of bends, and no doubt prevented many

fatal ones, but they still occurred. As the tunnels had many months to run at high air pressures, the question was: What else can be done to prevent them?

The writer, on coming out of the tunnel with the workmen, observed that the rate of decompressing was most irregular. One lock tender would allow the air to escape slowly until the 15 min. was almost exhausted, and then, by opening the valve, would let off the remaining pressure very quickly. Others would reverse the process, and exhaust quickly, and then keep the men under 2 or 3 lb. until the time expired. To avoid this, a simple decompressing valve was designed (Fig. 3) which gave a uniform decompression from 35 lb. to atmosphere in 15 min. A somewhat similar one (Fig. 4) was designed for the medical air-lock, for 1 hour decompression for 35 lb., with an automatic ventilator attached.

These valves certainly improved conditions, but still fatal cases occurred. After the first valve was under operation, the writer's attention was called to "Modern Tunnel Practice," by D. McN. Stauffer, M. Am. Soc. C. E., wherein a description is given of a needle decompression valve used on the air-locks at the Kiel Dry Dock Works, in Germany. A similar valve was also used for compression. On entering the lock the air was admitted at the rate of 1.5 lb. per min., and was decompressed at the rate of $\frac{3}{4}$ lb. per min., or, for 35 lb. gauge pressure, 23 min. for entering the air and 46 min. for leaving. This needle-valve was often frozen up, but otherwise worked well.

Such speeds seemed altogether too slow, and Mr. Stauffer in his book states that "these rates would be deemed excessively slow in American caisson practice." No date is given for the Kiel Dry Dock work, but presumably it was under way in 1904.

It was not thought advisable to increase the time of decompression at that time, but preliminary tests under air pressure in the tunnels for $1\frac{1}{2}$ hours were then tried for green men, followed by a second medical examination after decompressing in 15 min. A few men were eliminated by this test, and one case of permanent paralysis resulted from the test in 34 lb. gauge pressure. Fresh starters were made to stay in the tunnel for one-quarter of a shift for the first half, and if no caisson disease followed they were allowed to work for the second half. This proved a good safeguard.

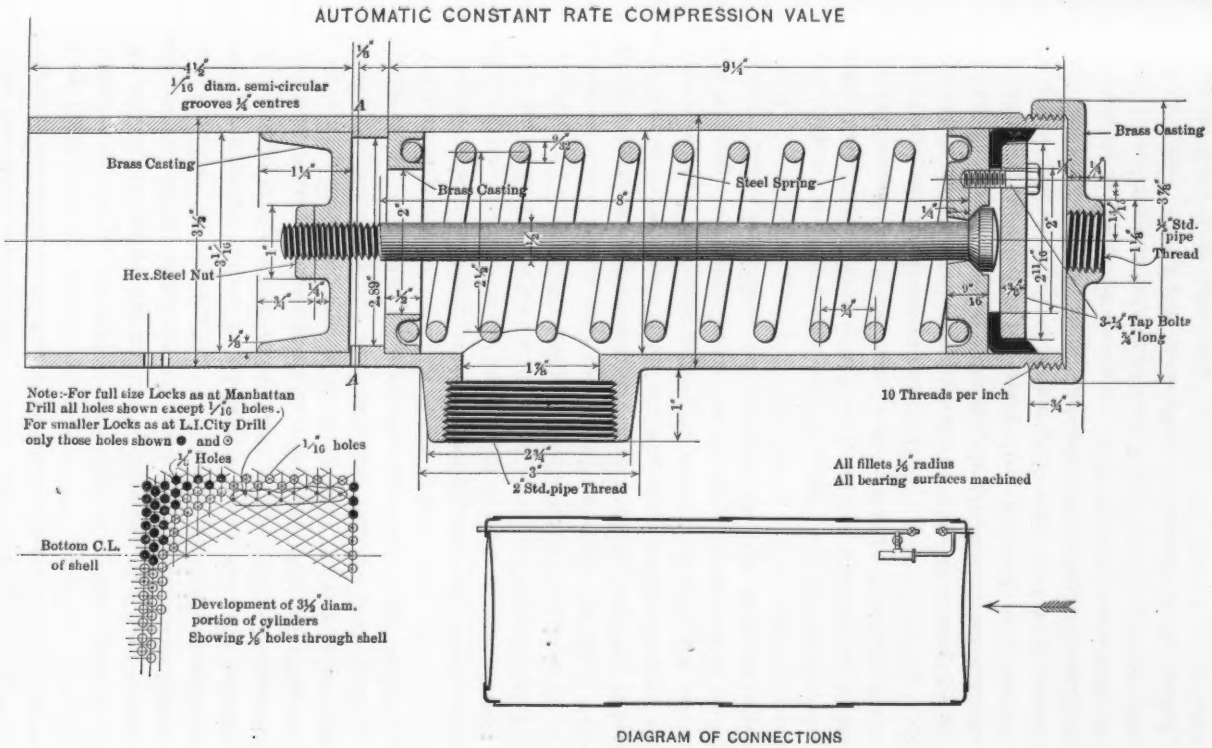
AUTOMATIC CONSTANT RATE COMPRESSION VALVE
4½"
1/16 diam. semi-circular grooves ¼" centres
1/8"
9¼"
A
A
Brass Casting
1¼"
Brass Casting
Steel Spring
Brass Casting
Hex. Steel Nut
3¾"
2 21/16"
1"
¾"
¼"
1/8"
2.89"
½"
2½"
2"
8"
¾"
¼"
2"
¾"
9/16"
¾"
¾"Std. pipe Thread
3-1/4" Tap Bolts ¾" long
10 Threads per inch
1⅞"
1"
2¼"
3"
2" Std. pipe Thread
Note:-For full size Locks as at Manhattan Drill all holes shown except 1/16" holes. For smaller Locks as at L.I.City Drill only those holes shown ● and ⊙
¼" Holes
1/16" holes
Bottom C.L. of shell
Development of 3½" diam. portion of cylinders Showing 1/8" holes through shell
All fillets ⅛" radius
All bearing surfaces machined
DIAGRAM OF CONNECTIONS

FIG. 3.

AUTOMATIC CONSTANT RATE DECOMPRESSION AND VENTILATING VALVE

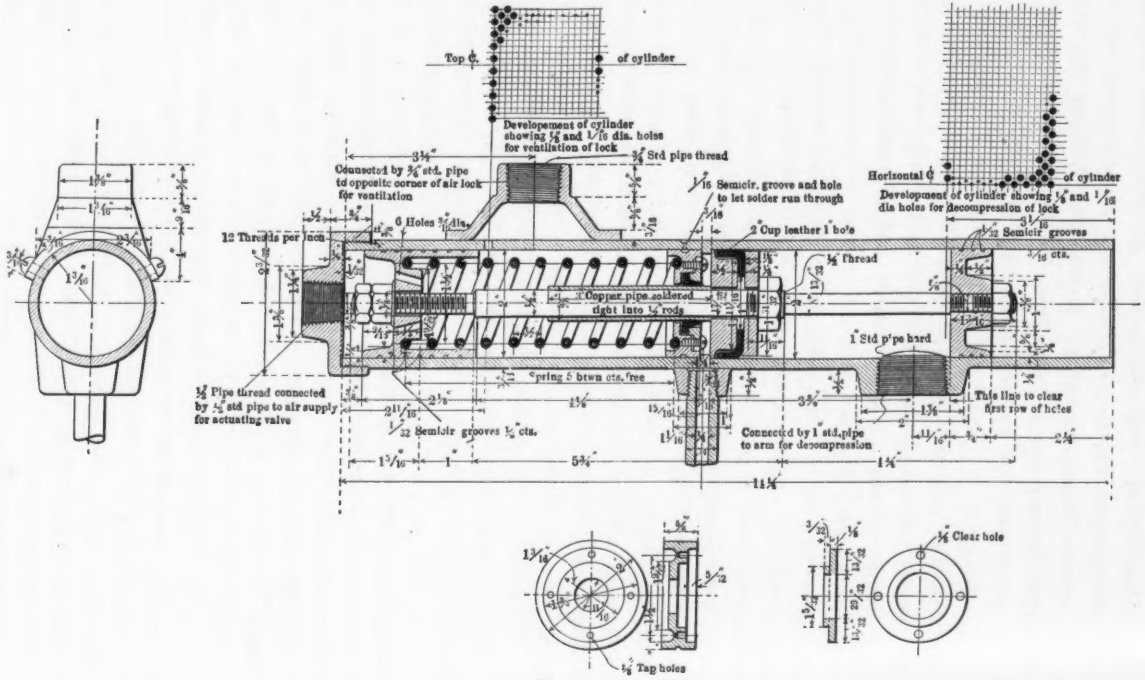

FIG. 4.

AUTOMATIC CONSTANT RATE COMPRESSION VALVE

On November 8th, 1906, a second bulkhead was put in operation in one of the tunnels, and the pressure between the two bulkheads was reduced to 15 lb. gauge. The number of cases of disease was very small for that tunnel, and as soon as possible additional bulkheads were placed in all four tunnels. The result was to have been expected from the experience in other tunnels where two bulkheads were used; the exercise of the long walk between bulkheads, at low pressure, seemed to assist in driving off the bubbles of air from the blood.

The workmen were allowed to decompress from 35 to 15 lb. as quickly as they pleased. They then walked for 500 ft. along the tunnel under 15 lb., taking at least 5 min., and then decompressed from 15 lb. to atmosphere in 10 min., so that in all about 16 min. were occupied in decompressing. When the inner pressure was less than 32 lb. an 8-hour shift was worked, with ½-hour interval for lunch, between bulkheads in low air pressure.

Just when it looked as if the double bulkheads with stage decompression had eliminated fatal and severe cases of caisson disease, two deaths occurred in physically perfect subjects.

In order to discover, if possible, the connection between the cases of disease and other things, charts were plotted by the medical staff for some months, showing the rise and fall of air pressure, hours worked, humidity, temperature, percentage of carbon dioxide, and number of green men in the tunnel, along with barometer readings, condition of weather, direction of wind, and number of cases of bends. The results were not very encouraging, but it was noted that the number of green men, the height of pressure in the tunnel and the number of cases of bends varied together.

The percentage of cases in air pressure of 31½ lb. for 8-hour shifts was no more than the percentage in 32½ lb. for two 3-hour shifts—in fact, it was, if anything, less for the longer shift. The decrease in length of shift added one extra gang of men, and probably many of these men being green accounted for this.

In November, 1906, Mr. Moir became acquainted in part with Dr. Haldane's work of investigation for the British Admiralty on "Deep Sea Diving," and ordered the decompressing valves changed so that the decompression was accelerated at the commencement of the operation, and slowed off toward the end. In October, 1906, the writer, endeavoring to find out the effect of sudden decompression,

fitted an ordinary glass siphon bottle with a valve and pressure gauge. This bottle was partly filled with water, and put under 36 lb. air pressure for 12 hours. On suddenly releasing the pressure, no effervescence whatever took place, and only a very few minute bubbles were visible. The same thing was tried with the bottle partly filled with bullock's blood, with like results. Arguing that the placid surface of blood in a stationary vessel offered no such opportunity for dissolving air as occurs in the lung surface with the blood circulating, the siphon was again charged with 36 lb. pressure, but was rotated for 24 hours under this pressure. The same result as before was observed on suddenly releasing the pressure; but, on closing the valve, the pressure recorded on the gauge was about 1 lb. per sq. in. in less than 1 hour, and in 6 days a pressure of 13 lb. was recorded, showing that the air dissolved in the blood came off very slowly.

If the cork of a bottle of aerated water is drawn quickly, it effervesces so violently that the water froths out of the neck of the bottle; but, if the gas is allowed to escape slowly, only a mild bubbling results. This in general is the theory on which slow decompression has been advocated in the past. The violence of the escape of carbon dioxide gas from aerated water, as compared with the escape of air from water released from pressure, is due to the greater solubility of carbon dioxide than air.

It was difficult to know what else could be done at this time in the way of eliminating caisson disease on the East River tunnels, and the cases of disease were certainly less frequent and less severe, although at long intervals fatal cases still occurred. The workmen, by this time, had become seasoned, and more seasoned men were now available from other works.

Dr. Leonard Hill read a paper on deep sea diving and caisson work, on September 26th, 1907, and exhibited the web of a living frog under a pressure of 20 atmospheres. On suddenly decompressing, bubbles were plainly visible in the blood vessels, multiplying quickly as they traveled along until the circulation was choked; on recompression the bubbles disappeared and circulation was resumed. He stated that he and Mr. Greenwood had been under an air pressure of 92 lb., and had suffered no ill effects.

Dr. J. S. Haldane, on November 29th, 1907, read a most enlightening paper before the Society of Arts, in London, in which he gave

the results of his investigation for the British Admiralty on "Deep Sea Diving."

He set forth very clearly the question of the solution of gases by the blood, fat, and tissues of the body, and attempted to determine the time taken to saturate the blood with the nitrogen of the air. He proved conclusively that the quantity of carbon dioxide adjusts itself to a constant percentage inversely proportional to the pressure in the alveoli, or air cells.

In atmospheric pressure, the percentage of carbon dioxide in the alveolar or expired air is 5.6%, and at a pressure of 2 atmospheres, absolute, this is reduced to 2.8%; so that the question of the percentage of carbon dioxide in the air of the working chamber is not important unless it approaches the percentage of carbon dioxide in the air cells of the lungs.

For instance, if the air-chamber is under an air pressure of 30 lb., or 3 atmospheres absolute, the percentage of carbon dioxide in the air cells is 5.6 divided by 3, or 1.86%; and, if the percentage of carbon dioxide in the air-chamber does not exceed 1%, no ill effects will arise. This is ten times as much as is generally specified, namely, 10 parts in 10 000, and greatly reduces the amount of compressed air necessary per man per hour, which can be calculated approximately from the following simple formula:

$$\text{Cubic feet per man per hour} = \frac{\text{80 cu. ft. permitted above normal air}}{\text{Percentage of rise of } CO_2}.$$

Thus, if 0.04% is the CO_2 in the atmosphere, and the percentage in the tunnel is allowed to go up to 0.10%, the air required per man per hour $= \frac{80}{0.06} = 1\,333$ cu. ft.

Dr. Haldane has completely removed all difficulty of respiration and discomfort of divers at great depths through the application of this theory regarding the percentage of CO_2 by supplying the diver with the same volume of compressed air per minute at different levels instead of the same volume of free air per minute.

The question of ventilating compressed-air tunnels seldom arises, as the volume of air generally far exceeds the amount required for purity, on account of the tremendous leakage at the face, so that this is not nearly as important as is indicated by Dr. Haldane's investigation of the solubility of air by the blood and tissues of the body.

DIAGRAM SHOWING RATE OF DESATURATION OF BLOOD AFTER VARIOUS PERIODS OF IMMERSION IN COMPRESSED AIR

FIG. 5.

After demonstrating that the CO_2 is dealt with automatically, he shows that nearly all the oxygen dissolved by the blood is taken up chemically by the hemoglobin, but the nitrogen remaining is incapable of combination with the system, and is dissolved by the blood and tissues. These, therefore, become saturated with nitrogen during immersion in compressed air.

If saturation takes place quickly it will not matter how long a workman stays under air pressure; but, if slowly, then shortening of the working shift will be a safeguard.

The rate of saturation is the same as the rate of desaturation if compression and decompression are instantaneous; so that, by altering the time for decompression, it is possible to determine by experiment the rate of saturation and desaturation.

Dr. Haldane experimented with men and goats at high air pressures, with varying lengths of stay under air pressure, and with different speeds of decompression. From these he concluded that certain parts of the body, where the circulation is rapid and the number of blood vessels high for the mass of the part supplied, would be half saturated or desaturated in 5 min., while other parts, with slight circulation for the mass, would require 75 min. for 50% saturation, especially the fatty parts, as fat is found to dissolve about six times as much nitrogen as blood.

Fig. 5 shows the rate of saturation, on the basis of half saturation in various times from 5 to 75 min., and curves of desaturation, if the decompression is instantaneous after varying times of immersion from ½ hour up to 8 hours.

It will be seen that 90% saturation takes place in the slowest parts of the body in about 4 hours, and complete saturation for the quickest parts in about 40 min., so that after 40 min. immersion there is danger in too rapid decompression.

Dr. Haldane has noted that no serious cases of caisson disease have occurred for rapid decompression at a working pressure of 19 lb. gauge pressure, or 2.3 atmospheres, absolute. Therefore, he concludes that it is always safe to decompress rapidly to half the absolute pressure, or even to the absolute pressure divided by 2.3, without bubbles being liberated. Further, if the decompression for the remaining pressure is continued at such a rate as will keep pace with the rate of desaturation, then, the absolute pressure of nitrogen in the blood never exceed-

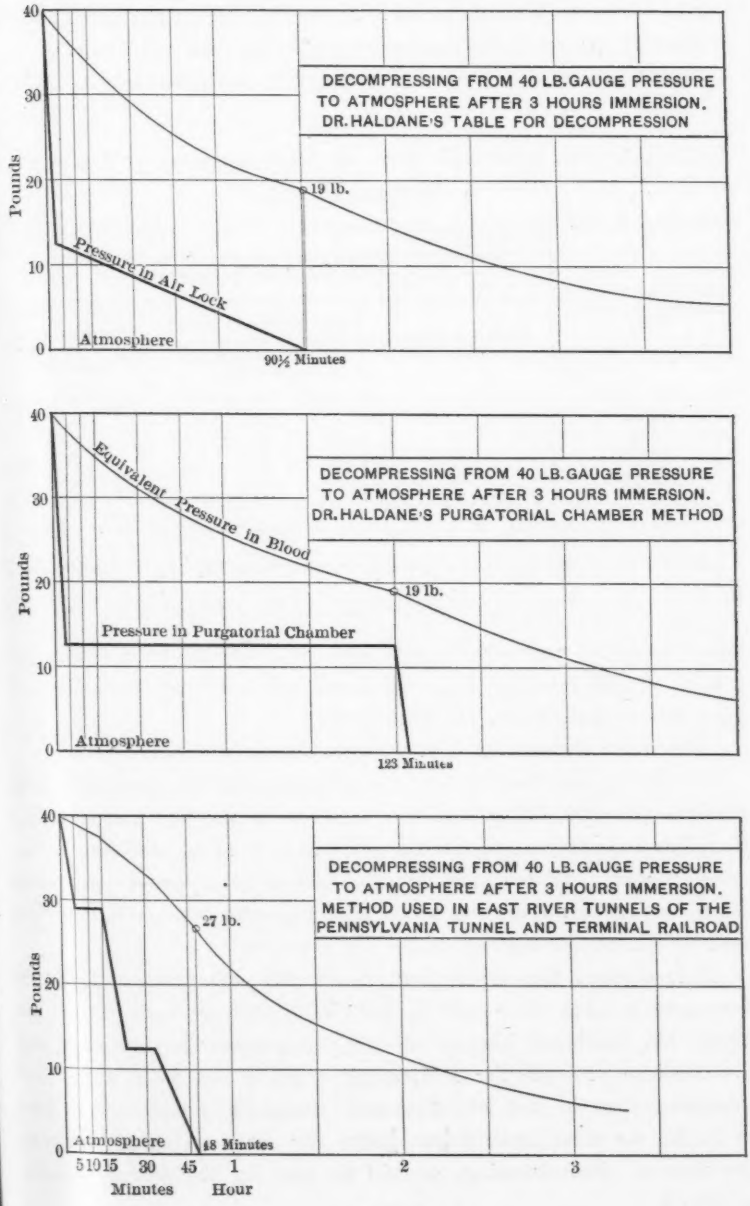
DECOMPRESSING FROM 40 LB. GAUGE PRESSURE TO ATMOSPHERE AFTER 3 HOURS IMMERSION. DR. HALDANE'S TABLE FOR DECOMPRESSION
Pounds
40
30
20
10
0
19 lb.
Pressure in Air Lock
Atmosphere
90½ Minutes
DECOMPRESSING FROM 40 LB. GAUGE PRESSURE TO ATMOSPHERE AFTER 3 HOURS IMMERSION. DR. HALDANE'S PURGATORIAL CHAMBER METHOD
Equivalent Pressure in Blood
19 lb.
Pressure in Purgatorial Chamber
Atmosphere
123 Minutes
DECOMPRESSING FROM 40 LB. GAUGE PRESSURE TO ATMOSPHERE AFTER 3 HOURS IMMERSION. METHOD USED IN EAST RIVER TUNNELS OF THE PENNSYLVANIA TUNNEL AND TERMINAL RAILROAD
27 lb.
Atmosphere
48 Minutes
5 10 15 30 45 1 2 3
Minutes Hour

FIG. 6.

ing 2.3 times the absolute pressure in the air-lock during decompression or after returning to the atmosphere, no symptoms will occur.

Table 1 shows the rates of decompression advocated by Dr. Haldane for caisson workers:

TABLE 1.—DR. HALDANE'S RATE OF DECOMPRESSION IN CAISSON AND TUNNEL WORKS.

Working pressure, in pounds per square inch.	NUMBER OF MINUTES FOR EACH PERIOD OF DECOMPRESSION AFTER THE FIRST RAPID STAGE.		
	After first 3 hours' exposure.	After second or third 3 hours' exposure showing an interval for a meal.	After 6 hours or more of continuous exposure.
18-20	2	3	5
21-24	3	5	7
25-29	5	7	8
30-34	6	7	9
35-39	7	8	9
40-45	7	8	9

If the pressure is 40 lb., gauge, decompress rapidly from 40 lb. to $\frac{40+15}{2} = 27\frac{1}{2}$ lb., absolute, or $12\frac{1}{2}$ lb., gauge, in 3 min., and then take 7 min. for each remaining pound, viz., $12\frac{1}{2}$ lb., or $87\frac{1}{2}$ min. plus 3 min. = $90\frac{1}{2}$ min., or $1\frac{1}{2}$ hours in all for a 3-hour immersion; or, for a 6-hour immersion, 115 min. in all.

The upper diagram on Fig. 6 shows this graphically.

It will be seen that, if this rule is followed, the condition of the absolute pressure, being half the pressure in the blood, is obtained. If uniform decompression for the same time were adopted, the decompression would be too slow for the earlier part, would very much retard the desaturation, and would give a greater tension in the blood than 19 lb., on coming out of the air pressure.

It is obvious that while the air-lock has any pressure in it the desaturation takes place only in relation to that pressure, and, if one adopts Dr. Haldane's idea of keeping a portion of the tunnel at half the absolute pressure for a dressing room to act as a purgatorial chamber, where the men may wash and change while desaturating down to 19 lb., we must make a new curve for desaturation and increase the time of decompressing, as will be seen by the second diagram on Fig. 6.

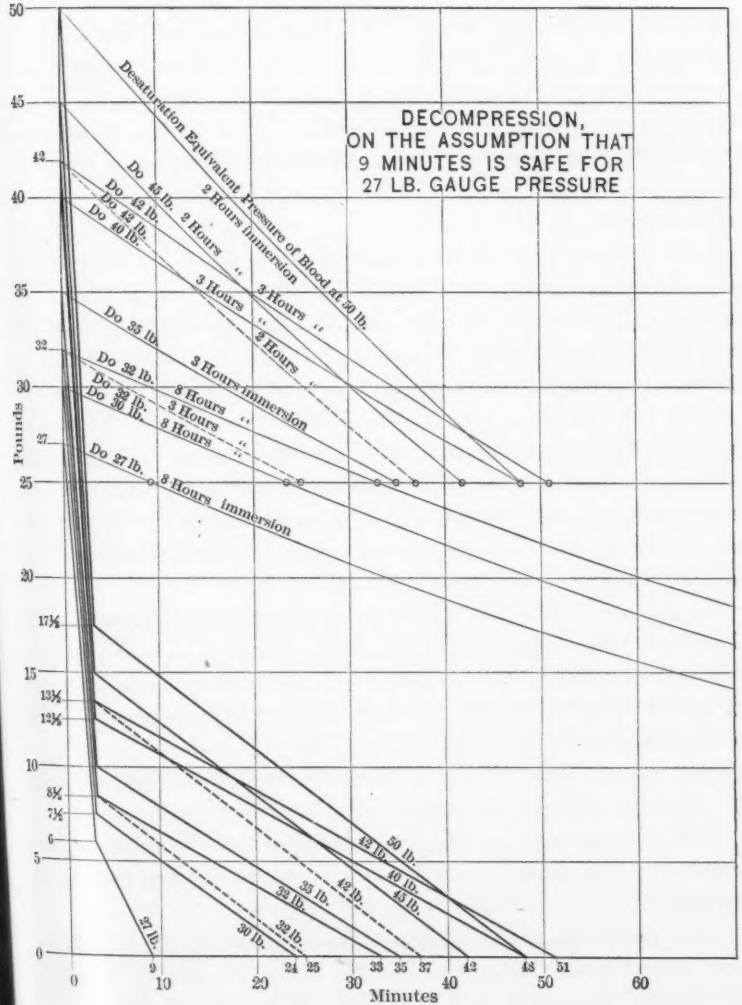
DECOMPRESSION,
ON THE ASSUMPTION THAT
9 MINUTES IS SAFE FOR
27 LB. GAUGE PRESSURE
Desaturation Equivalent Pressure of Blood at 50 lb.
Do 45 lb. 2 Hours immersion
Do 42 lb. 2 Hours "
Do 42 lb. 3 Hours "
Do 40 lb. 3 Hours "
2 Hours "
Do 35 lb. 3 Hours immersion
Do 32 lb. 3 Hours "
Do 32 lb. 3 Hours "
Do 30 lb. 3 Hours "
Do 27 lb. 3 Hours immersion
50 lb.
42 lb.
40 lb.
43 lb.
42 lb.
35 lb.
33 lb.
32 lb.
30 lb.
27 lb.
Pounds
Minutes
50
45
42
40
35
32
30
27
25
20
17½
15
13½
12½
10
8¼
7½
6
5
0
0
9
10
20
24
25
30
33
35
37
40
42
43
50
51
60

FIG. 7.

From this it appears that the tension in the blood does not reach 19 lb. until a period of 123 min. has been passed in the purgatorial chamber after 3 hours' immersion.

It will be noticed that Dr. Haldane bases his theory of decompression on the fact that no cases of caisson disease are chronicled for men working in gauge pressures up to 19 lb., or 34 lb., absolute, and if it is safe to decompress suddenly from 34 lb., absolute, to 15 lb., absolute, it is safe to decompress suddenly from 50 lb., absolute, to 22 lb., absolute, or from 35 lb., gauge pressure, to 7 lb., gauge pressure.

The question arises at once: Are the times suggested by Dr. Haldane practical?

Many caissons of small dimensions are sunk under air pressures of 35 lb. and to cramp men in a small air-lock for 73 min. is out of the question, and the fitting of large compartments in small vertical air-locks, as used in building foundations in New York City, would be difficult. In tunnel work it would be easier, but the present method of rapid decompression is so radically different from Dr. Haldane's suggestion that it quite appals one to think of taking so long.

The experience of most compressed-air works is that up to 27 lb. gauge pressure there is very little trouble, and using this as the safe limit the times required to reduce the saturation in the blood to 27 lb. is not so excessive for pressures up to 50 lb.

The horizontal lines on Fig. 5 show the equivalent points for 27 and 19 lb. in the blood for 50, 45, 40, 35, 30, and 27 lb., gauge pressure.

It will be noticed that the slowly saturated parts of the body are only partly saturated in the first few hours of immersion, and therefore the desaturation is proportionately fast.

From Fig. 5, Tables 2 and 3 have been compiled; they show the risks taken in rapid decompression after long immersion.

If the decompression is slow, these curves will be flattened out, and a longer time will elapse before the saturation of the blood falls to the equivalent of 19 lb. or 27 lb., as the case may be.

Fig. 7 shows the curves of decompression from 27, 30, and 32 lb., gauge, after 8 hours' immersion, and for 32, 35, 40, and 42 lb., gauge, after 3 hours' immersion, and 42, 45, and 50 lb., gauge, after 2 hours' immersion, on the basis that it is safe to decompress from 27 lb., gauge pressure, in 9 min. This reduces the saturation in the blood to the equivalent of 25 lb. on reaching atmospheric pressure.

TABLE 2.—TIMES (IN MINUTES) REQUIRED FOR PRESSURE IN BLOOD TO FALL TO THE EQUIVALENT OF 19 POUNDS GAUGE PRESSURE WITH INSTANT DECOMPRESSION AFTER VARIOUS PERIODS OF IMMERSION IN COMPRESSED AIR.

No. of hours of immersion.	GAUGE PRESSURES:					
	27 lb.	30 lb.	35 lb.	40 lb.	45 lb.	50 lb.
	Minutes.	Minutes.	Minutes.	Minutes.	Minutes.	Minutes.
½ hour	4	5	8	10	14	16½
1 "	7	10	15	20	26	32
1½ hours	10	16	24	32	40	48
2 "	14	21	32	42	52	62
3 "	22	30	45	59	72	82
4 "	28	39	56	69	82	93
5 "	33	44	60	74	86	98
6 "	35	47	63	76	89	100
7 "	37	48	64	78	90	102
"	38	50	66	79	92	103

The times in Table 2. being for instant decompression, must be suitably extended for slow decompression.

TABLE 3.—TIMES (IN MINUTES) REQUIRED FOR PRESSURE IN BLOOD TO FALL TO THE EQUIVALENT OF 27 POUNDS GAUGE PRESSURE WITH INSTANT DECOMPRESSION AFTER VARIOUS PERIODS OF IMMERSION IN COMPRESSED AIR.

No. of hours of immersion.	GAUGE PRESSURES:					
	27 lb.	30 lb.	35 lb.	40 lb.	45 lb.	50 lb.
	Minutes.	Minutes.	Minutes.	Minutes.	Minutes.	Minutes.
½ hour	.....	1	3	4	6	8
1 "	.....	1½	5	8	12	16
1½ hours	.....	2	6	12	18	24
2 "	.....	3	9	17	25	32
3 "	.....	5	15	25	34	45
4 "	.....	6	19	32	45	57
5 "	.....	8	22	37	50	61
6 "	.....	9	25	40	52	64
7 "	.....	10	27	41	54	65
8 "	.....	11	28	43	55	66

The times in Table 3, being for instant decompression, must be suitably extended for slow decompression.

Table 4 is taken from Fig. 7, and in part is confirmed by the experience in the East River tunnels as comparatively safe. Caisson disease will result, but no fatal or severe case should be experienced with physically sound men if these times are adhered to.

TABLE 4.—DECOMPRESSION TABLE BASED ON 9 MINUTES BEING SAFE FOR 27 POUNDS GAUGE PRESSURE.

Gauge pressure, in pounds.	Reduce pressure in 3 minutes to:	Total time in air-lock after 8 hours' work:	Total time in air-lock after 3 hours' work:	Total time in air-lock after 2 hours' work:
27	6 lb.	9	....	....
30	7½ "	24	....	....
32	8½ "	33	25	....
35	10 "	....	35	....
40	12¼ "	....	48	....
42	13½ "	....	51	37
45	15 "	....	....	42
50	17½ "	....	....	48

Some time after reading Dr. Haldane's paper and studying his theory, it became necessary to raise the pressure in the tunnels to 40 lb., gauge. It was possible to make the workmen pass through three sets of air-locks on leaving the tunnel. The inner chamber was kept at 40 lb., the intermediate chamber at 29 lb., and the outer chamber at 12½ lb.

The men were ordered to take 5 min. in the first lock, 8 min. in the second lock, and 15 min. in the third. There was a distance of approximately 1 000 ft. between each pair of locks. Walking this distance and gathering in the stragglers generally required 10 min. to each chamber, so that, in all, 48 min. were taken to decompress from 40 lb. to atmosphere. No severe or fatal cases resulted, and little time was lost by the men through caisson disease, the cases being only slight. Under this pressure 330 men were employed for 36 days, working 3 hours on, 3 hours off and 3 hours on. It is true that no green men were used on this work, as there were plenty of experienced air men available at that time.

The third diagram on Fig. 6 shows the desaturation curve and rate of decompression, leaving the saturation of the blood equivalent to 27 lb. on reaching atmospheric pressure. This result bears out in part the periods of decompression of Table 4, and, if a more rapid passage had been made through the first and second locks, probably a better result would have been attained, as the equivalent pressure of the blood would have reached 25 lb. on coming out, as shown by Fig. 7, instead of 27 lb.

In caissons or tunnels with but one lock, it is a difficult problem to allow workmen as long a time as Table 4 indicates, as no one can enter the air-lock during decompression. One method of overcoming this difficulty would be to provide a lock with two small end chambers

and a larger center chamber, four doors in all being necessary. Anyone making a short visit to the caisson could pass through without disturbing the pressure in the middle decompressing chamber. Fig. 8 is a diagram of such a lock with connections.

Some one has suggested, for diving bells and caissons, a detachable chamber like a boiler which the men could enter, and thus take as long as necessary to decompress.

AIR-LOCK WITH MIDDLE DECOMPRESSING CHAMBER

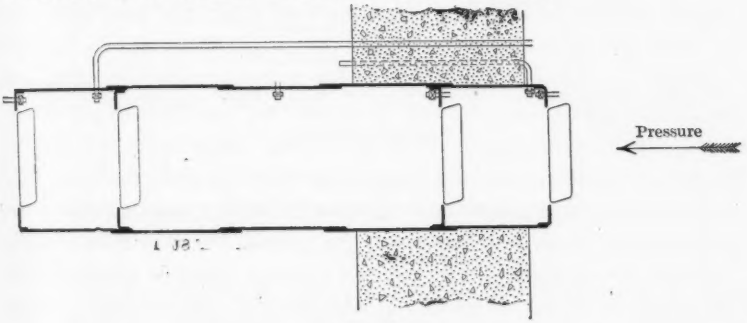

FIG. 8.

The men on the East River tunnels rebelled against 15 min. for decompression, but, after putting the responsibility up to the foremen, in time they found that it was a safeguard and voluntarily lengthened the time to 20 min., and gladly submitted to 48 min. for 40 lb.

The death rate due to caisson disease was comparatively small, averaging $\frac{19}{100}$ of 1% for the whole of the compressed-air work, and, from the experience gained, it would in all probability have been much higher if the decompression had not been lengthened.

The fact that the only recognized cure for caisson disease is recompression in a medical air-lock followed by slow decompression, is a powerful argument in favor of slow decompression, and where it is at all possible, in future works, regulated decompression will in all probability be adopted.

In caissons with small air-locks, the volume of air remaining when the lock is full of men is very slight, and very rapid decompression takes place. The workmen have a good opportunity to become seasoned in a caisson, as the pressure begins at 1 or 2 lb. and gradually increases day by day as the caisson sinks, and the highest pressures are required for but a few days.

On the East River tunnels two caissons were sunk to a final pressure of 33½ lb. and very few cases of caisson disease occurred, none of which was fatal although the decompression was rapid. On the other hand, on account of the tunnels on the Manhattan side starting out at a high pressure, the men had no chance to get seasoned, and many cases occurred, though the time of decompression was regulated, but not to such an extent as Table 4 indicates.

Thus far, practically, no grandfatherly legislation hampers engineers in the United States in compressed-air work, but such legislation is threatened, and engineers should be prepared to guide it. The engineer is confronted with conflicting testimony on all sides, and, though his experience shows that some men are capable of rapid decompression without injury, he also knows that many men are injured. Because a few men can for a time rashly take the risk, shall he eliminate good men from compressed-air work by putting them to such a test without endeavoring to make it safe for all who are physically sound?

Prevention is better than cure, but absolute safety is impracticable, and some risk must be taken. In the writer's opinion, the hours of labor and time of decompression are matters that should receive the attention of engineers. The effect of legislation formulated by men ignorant of the conditions might greatly embarrass construction work, and, although it is contrary to the custom of this Society to draw up rules and regulations, yet it might render valuable aid if a committee of engineers, contractors, and doctors could be convened to study this subject and collect data, and ultimately draw up a table which would be a guide to the Profession. Such a table would be no doubt a compromise between the time worked in the air pressure and the time taken to decompress. It would be a statement of a body of men in high repute; if acted upon, it would relieve construction companies of liability to a great extent, and, if carefully formulated, it would greatly lessen the dangers of compressed-air work.

The writer is indebted to Alfred Noble, Past-President, Am. Soc. C. E., Chief Engineer of the East River Division of the Pennsylvania Tunnel and Terminal Railroad; and to E. W. Moir, M. Am. Soc. C. E. Vice-President of S. Pearson and Son, Inc., for permission to include such information as is taken from the work of the East River tunnels; and to Dr. Haldane for permission to make use of Table 1, and, generally, for the benefit of his theories, in writing this paper.

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AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

THE SIXTH STREET VIADUCT, KANSAS CITY.
Discussion.*

By Messrs. Daniel Bontecou, and Victor H. Cochrane.

Mr. Bontecou.

Daniel Bontecou, M. Am. Soc. C. E. (by letter).—This paper presents many points of interest, and is well worth careful reading.

Since the author gives some details of the preliminary investigation of probable profits, it would be of interest to know more of the assumptions made as to the proportion of the total team traffic which could be depended on to use the viaduct, and of the extent to which they were realized. The estimates of cost and the operating expense given would mean that the gross earnings of the completed structure would need to be about $900 per day. The revenue from street-railway traffic could be determined fairly well, but could hardly be estimated at more than, say, $125 per day, leaving a very large proportion of the earnings to be derived from team traffic. In view of the risk in estimating the use which would be made of a utility of this kind, where it is optional to pay a toll or to take another and slightly less convenient route, it is a fair question whether so expensive a structure should have been built otherwise than by the communities served; or, whether a cheaper structure should not have been built to serve the street-car traffic only, in which case no operating expense except maintenance would be involved.

Since all the cars, and probably most of the other vehicles, must meet grades steeper than 1½%, it would seem that the viaduct roadway might have been placed with advantage nearer the surface, and still have afforded the proper clearances.

* This discussion (of the paper by E. E. Howard, Assoc. M. Am. Soc. C. E., printed in *Proceedings* for February, 1909), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

Mr. Bontecou.

The shore line of the Missouri River at the site of the trestle on the accretion was about 600 ft. south of the structure only some twelve years before the latter was built, and has been regulated by protection work a few miles above, and also near the mouth of the Kaw. The design of the trestle foundations, therefore, involved a careful consideration of the permanence of the river regulation. The experience of the 1903 flood seems to the writer to justify the choice of pile foundations, but the conditions of soil and loading appear to call for piles longer than the concrete piles described.

In building the foundations for some large engines in similar ground near the viaduct, tests on a surface of 2 sq. ft. at a depth of about 15 ft. showed a supporting power of 1 340 lb. per sq. ft., and a settlement of 0.3 ft. under a load of 2 800 lb. per sq. ft. It was necessary to use oak piles only, 22 to 24 ft. long, and 300 were driven in an area of 2 100 sq. ft.—the intention being to depend on the combined resistance of the piles and the compressed sand, silt, and clay. After allowing for the frictional resistance of the supported concrete, the load amounted to 36 000 lb. per pile, or 4 480 lb. per sq. ft. of foundation, the value of the piles, by the Wellington formula, being 30 000 lb. This foundation was deeply submerged in 1903 without the occurrence of settlement.

In cases where piles are dependent on friction, in somewhat plastic clay, there is probably an element of safety in driving them with a heavy hammer and low fall, rather than jetting them to place, and in arranging them as far as practicable so as to confine and compact the soil. The concrete piles used in the viaduct had a surface area of 130 sq. ft., and, as placed and tested, showed a resisting power of 38 000 lb., or rather less than might be expected from piles driven in a cluster in such material, and probably less than would be desirable if the roadway were actually loaded to 100 lb. per sq. ft.

To determine the effect of clay and dirt on the strength of concrete a good many tests have been made, with varying results; and, although the writer has often permitted an admixture of 5% of earthy material, he has preferred usually to wash dirty stone in barrows having open-mesh bottoms until the water ran clear. It is difficult to understand why the presence of clay should add anything to the strength of concrete, as shown by the author's tests, and if stronger concrete can be made of well-mixed and graded materials with clay, than without it, the writer must confess to a great deal of misdirected energy in the past.

The Kaw River spans, as described, are an interesting example of riveted construction. In view of the high ratio of dead to live load, it would seem to many that a smaller impact allowance was permissible for the motorway load, and the well-understood advantages of pin connections seem to apply to this case with special aptness.

A comparison of the weight and cost of the adopted design with

one based on three pin-connected trusses for each span with cross-beams carried entirely above the top chord would be an interesting addition to the paper. Mr. Bontecou.

Ever since the flood of 1903 there has naturally been a feeling of apprehension regarding the Kaw River, and the requirement by the War Department that bridge piers be carried to bed-rock has been cheerfully accepted, regardless of the fact that, while fifteen local bridges were destroyed by that flood, there was no case in which a well-constructed pile foundation failed. It is not apparent, therefore, why it should have been thought necessary to excavate the bed-rock to receive any part of the pier construction.

VICTOR H. COCHRANE, ASSOC. M. AM. SOC. C. E. (by letter).—The writer left the office of the Consulting Engineers to take charge of the shop inspection. In some respects the work was unique, and a brief account of it, in addition to what the author has given in his unusually complete paper, may be of interest. Mr. Cochrane.

There were three inspectors on the work, two employed by the Consulting Engineers, and one by the manufacturers of the paint specified for the structure. While in the office, the writer, in the capacity of Chief Draftsman, had assisted in designing the main portion of the viaduct, and was familiar with the locality where the structure was to be built; consequently, he was well acquainted with the theoretical and practical considerations affecting the design. Accordingly, he was given authority to approve all shop drawings. As the author remarks, much time was saved by this arrangement, the drawings being put in the shops, when desired, almost immediately after being turned over to the writer for approval. The chief saving in time, however, resulted from the advice and assistance the writer was able to give to the force engaged in preparing the shop drawings. It was necessary in several instances to make considerable modifications in the design, and these changes were made without having to await the approval of the Consulting Engineers. The writer's experience in this case convinces him that the inspector in charge of a piece of work of this magnitude should be quite familiar with the design, in both its theoretical and practical aspects.

The squad of draftsmen assigned for the shop detailing seemed to consist largely of men inexperienced in making drawings for such a structure; and, although the engineers' plans were quite complete, the first shop drawings finished and checked were found to have so many errors that they had to be practically re-checked before being approved. After the men became more experienced in the work, it was not necessary to check the drawings so much in detail. Altogether, there were more than 600 sheets. From time to time, after a number of drawings had been approved, the weight of each member was computed and entered in a quadrille-ruled notebook.

Mr. Cochrane.

The paint used on the work was guaranteed by the manufacturers for a period of ten years, provided they were allowed to employ inspectors to watch the work in all stages. Their representative in the shops succeeded in getting excellent work, although there was much friction at first. The paint used in assembling was so heavy and stiff that there was considerable difficulty in applying it. The painting was done under contract by Greek laborers, and, at first, it was difficult to get them to clean the metal as thoroughly as desired, since the standard adopted was much in advance of anything to which they had been accustomed; the quality of work required, however, was clearly and forcefully stated in the specifications, and, in cases of dispute, the paint inspector was nearly always sustained by the representative of the engineers.

The punching, assembling, and reaming was carefully watched throughout. Each member after being finished—but before being painted—was inspected for loose rivets and other defects, and all leading dimensions and open-hole spacings were carefully verified. In some kinds of members many pieces were nearly alike. In such cases lists were prepared and used instead of the unwieldy drawings. The dates of acceptance were noted on these lists, or on the prints in case they were used. Many pieces had to be sent back for correction, as was to be expected, but no very serious errors were found, and little material was rejected on account of surface defects. The provision for field reaming greatly facilitated the shopwork, but increased the labor of checking up field connections, as no assembling was done, and consequently there was no way to insure the matching of rivet holes except by measuring each connection.

In cases of errors the inspectors endeavored to remedy the defect in such a way as to cause the least possible expense to the shops without impairing the strength of the piece. When the shop employees learned that the inspectors could be depended on to assist them in correcting mistakes, little or no attempt was made to conceal them, and they were usually brought to the attention of the inspector as soon as discovered.

Each day the inspector entered in a scratch-book, without regard to order, the marks and number of pieces of all material inspected during the day. As soon as the invoices of shipments were received, the material listed on the invoices was checked off the scratch-book; at the same time the scale weights were compared with the calculated weights. By this means many mistakes were discovered in the invoices; some were due to misreading the marks painted on the pieces; in many cases, the scales were incorrectly read; and, in some instances, the number of members shipped was in error. The scratch-book was not intended as a permanent record, but each Saturday afternoon a list was made of unfinished work then in the shops, a column headed "Inspected" being filled out from the scratch-book. Unfinished pieces

remaining in the shops from week to week were not re-entered in this book unless advanced in construction, in which case they were checked off in a column headed "Remarks." Thus the book showed at a glance the exact state of unfinished work at any time. Mr. Cochrane.

A record of shipments was made from the corrected invoices once each week in a book, a column headed "Number Previously Shipped" giving the numbers taken from a previous record-book. Owing to the errors in the invoices, previously mentioned, the shop's records of shipments, after a time, became very unreliable, and it was necessary to depend on the inspector's records altogether.

In preparing the record-books mentioned, the headings of the various columns were not repeated on each page, but were placed near the beginning and end of the book, the tops of the intervening leaves being cut away.

The difference between the estimated and scale weights for the main viaduct was only three-tenths of 1 per cent.

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AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

PROGRESS REPORT OF SPECIAL COMMITTEE ON CONCRETE AND REINFORCED CONCRETE.

Discussion.*

By Wilbur J. Watson, M. Am. Soc. C. E.

Mr. Watson.

Wilbur J. Watson, M. Am. Soc. C. E. (by letter).—As stated in the Minority Report of this Committee, the Majority Report, or much of it, is based on arbitrary rules for designing and executing works in reinforced concrete.

Such a set of rules should be based on sound theory, which represents the best current practice, and should be thorough and complete. It appears to the writer that these three requirements have been violated in a few important particulars. In criticizing such a report as this, it should be borne in mind, however, that we are dealing with a material, or rather a combination of materials, the use of which in engineering construction is comparatively new, and the experimental data on which, while complete enough in some ways, are very meager in others. Consequently, much confusion exists among designers with regard to the rules which they use for designing and executing in this material.

Viewed as an effort to standardize such rules and practice, the report of the Committee is commendable. A report of this kind, by a committee of this Society, is generally accepted by the majority of engineers in the United States as representing the last word on any disputed point in engineering practice; and such reports should not contain any statements which cannot be backed up with conclusive

* This discussion (of the Report printed in *Proceedings* for February, 1909), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

Mr. Watson.

experimental data or rigid mathematical analysis, and, in the writer's opinion, should contain no rules which must be considered as only tentative.

In regard to the corrosion of metal reinforcement embedded in concrete, the writer is somewhat disappointed to find no reference to the danger of corrosion of such steel when used as tensile reinforcement in beams with such high working stresses that serious tension cracks occur in the surrounding concrete.

The report states that concrete is sensitive to temperature changes, but does not state to just what extent. This information is needed, and would certainly be highly appreciated. It is the writer's notion, based upon observation of numerous concrete arch structures, that concrete is not sensitive to temperature, as compared with metallic structures. In fact, this statement of the report is partially contradicted in the next article, where, under the caption "Fire-Proofing," the statement is made that "the low rate of heat conductivity of concrete is one reason of its value for fire-proofing."

The article on shear and diagonal tension appears to the writer to be open to serious criticism. The action of shear members in providing against diagonal tension due to shear stresses is now pretty well understood. It is an established principle that, when the tensile stresses in a reinforced concrete beam exceed the allowed tensile stresses in the concrete, the metal reinforcement must be depended on to take care of all the tensile stresses without any reliance on the tensile strength of the concrete.

Precisely the same reasoning applies to diagonal tensile stresses, and, to be consistent, whenever the diagonal tensile stresses exceed the allowable tension in the concrete, metallic web members should have sufficient section to take care of the entire diagonal tension. It seems to the writer that it is unnecessary to give such indefinite methods as the Committee recommends for providing against shearing stresses. Such stresses are readily computed, and the necessary web members to provide for their resistance can be easily proportioned.

In Article VIII, Section 4, a working stress of 650 lb. for concrete in compression due to bending is recommended, but no reference is made as to whether such bending stress is to be computed by the "straight-line" or "parabolic" methods, which are both in use and with results varying by about 15 per cent.

The bonding stress of 80 lb. per sq. in. allowed in Section 6 is certainly much greater than is now used by the majority of conservative designers, who do not exceed 50 to 60 lb.

In Appendix B, the symbol P is used to denote the stresses in a single reinforcing member, while the same symbol is used to denote the total safe load on a column. The writer can see no reason for reducing the area of diagonal web members to seven-tenths of that

Mr. Watson.

required for vertical tension members, but recognizes the theoretical advantage of the diagonal web members.

The recognition of the desirability of continuous beam construction is to be commended, but the formulas are not those in common use, and the writer questions the wisdom of reducing the center moment below eight-tenths of that for a simple beam, as now commonly used.

On the other hand, no provision whatever is made for the increase in the shearing stresses due to continuity, and this appears to be a serious omission. The writer increases shears for continuous construction by 25% over that for simple beam construction.

This whole matter of continuity requires very careful consideration, and approximate methods of analysis should not be encouraged any more than is necessary. Most beams in reinforced concrete construction are T-beams, and the distribution of moments between the sections and the supports is far different from that in rectangular homogeneous beams, the moment at the supports being ordinarily decreased, and those in the center being increased, by reason of the T-section and the non-homogeneity of the material. However, very few designers provide sufficient reinforcement over the supports. In many cases, the writer has found that, with ordinary T-sections and percentages of reinforcement, using a center moment for uniform loads equal to $\frac{W l^2}{10}$, slightly more than two-thirds as much reinforcement should be provided over the supports as is provided at the centers, and care should be taken not to overstress the concrete in compression on the under side of the beam next to the supports.

It might also be well to call attention to the fact that practically all columns in reinforced concrete construction are subject to more or less severe bending stresses, owing to the fact that, while the dead load is often symmetrical about the column, the live load seldom is, and the inequality in the distribution, in monolithic construction, of the live load always produces bending moments in the columns. It would seem to the writer, therefore, that it is never safe to omit the longitudinal reinforcing rods in columns of this class.

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PAPERS AND DISCUSSIONS

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STEEL SHEETING AND SHEET-PILING.
Discussion.*

By Messrs. Charles W. Sherman, C. C. Conkling, Charles H. Higgins, Charles Evan Fowler, and J. C. Meem.

Mr. Sherman.

Charles W. Sherman, M. Am. Soc. C. E. (by letter).—As an economical application of steel sheet-piling, the author mentions its use, in connection with dams or embankments, as a core to prevent seepage, and refers to two or three cases. While it is unquestionably true that, in original construction, such a method of making a tight core is both feasible and cheap in many cases, the writer has never been able to satisfy himself of the durability of such a core. Most embankments which serve to retain water are more or less saturated, and the steel sheet-piling, therefore, would be constantly in contact, throughout at least a part of its length, with moist earth, and rapid corrosion of the steel would seem inevitable under these circumstances. The rate of such corrosion would doubtless depend on the precise chemical composition of the steel and the character of the soil with which it was in contact. It hardly seems likely that, in ordinary cases, such a core would last longer than six or ten years, even though comparatively heavy steel sections were used.

Mr. Conkling.

C. C. Conkling, M. Am. Soc. C. E. (by letter).—The writer wishes to thank Mr. Gifford for bringing out some of the practical points in the design and use of steel sheet-piling. Having taken considerable interest in the subject, during the past two years, and having discussed it in general with many engineers and contractors throughout the country, he finds that, while they nearly all recognize the merits

*This discussion (of the paper by L. R. Gifford, Assoc. M. Am. Soc. C. E., printed in *Proceedings* for February, 1909), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

Mr. Conkling.

of steel sheet-piling and its great advance over the old methods of using wood, still there is a great deal of misunderstanding regarding the requisites of a steel sheet-pile, and of the work which it has to perform during and after its installation.

Observations, covering the plans and installations of this form of piling, under many varying conditions, lead the writer to agree with Mr. Gifford that the primary quality necessary in the steel sheet-pile is its ability to resist (as a beam) earth or water pressure, either as a beam supported at the ends, as a continuous beam supported at several points, or as a cantilever beam where no bracing is used. In the first two cases it should do this with the least possible amount of bracing. In the case where the pile acts as a cantilever beam, it resists these pressures without the assistance of cross-bracing, or mainly by its resistance against bending at or near its point of penetration, and the aid which it receives by being interlocked with the adjacent piles.

This applies where the piling is used without bracing, in the construction of low single walls or circular or rectangular pockets of shallow depth. After excavation, such pockets are wholly or partly filled with concrete, clay puddle, stone, gravel, etc., in order to form units in foundations, large coffer-dams, dock walls, and other temporary or permanent constructions.

Where the piling is used as a core-wall or curtain-wall in connection with earth embankments, such as levees, dams, etc., it assists against sliding, and, in the case of dock walls, against sliding and overturning; such conditions also produce a cantilever action. In the case of walls of various kinds, the pile may also assist in giving vertical support, as a bearing pile, by friction against the earth throughout the length of its penetration. In any of these latter cases the ultimate strength of the steel wall against horizontal or oblique pressure is largely dependent on the strength of the interlocked joint of the steel piling.

Such pressures brought upon an unbraced steel sheet-pile wall, especially in its upper portion above the point of penetration, tends to bow the wall and produce tensional and transverse stresses on the interlocked joints, making the strength of these joints one of the most important factors of the steel sheet-piling.

A theoretical steel sheet-pile would seem to be one in which the strength of the interlocked joint, under tensional or transverse stress, would be equal to the strength of the web of the pile under similar stress; taking into consideration, at the same time, the lightest possible weight which will give enough metal in the pile to produce sufficient stiffness to allow it to be driven to final position, and afterward to perform its duty as a beam.

At the present time, as far as the writer has investigated, there

is no form of steel sheet-piling which exactly meets all these theoretical requirements. Mr. Conkling.

To illustrate, in part: Take a pile having a web ½ in. thick, allowing the ultimate strength of the steel in tension to be 60 000 lb. per sq. in. In a piece of piling 1 in. long, a web of this thickness would have a sectional area of ½ sq. in., and an ultimate strength in tension of 30 000 lb. The joint of two such pieces, when interlocked, should then show an ultimate tensional strength, in direct pull, of 30 000 lb. per lin. in. of joint, thereby equaling the strength of the web.

In physical tests of a great many forms of piling, by pulling to destruction in an Olsen testing machine, the writer has noted joints showing 32⅓% of the web strength, or 9 700 lb. per lin. in. of joint, as against ½ sq. in. of web, this being noted as the nearest approach to the conditions mentioned. He has also noted interlocking joints where the same test showed only 4½% of the web strength, which results indicate the great difference in the design and strength of some of the present forms of interlocking joints. These results might indicate that possibly, in some cases, the strength of the interlocking joint has been overlooked, or made secondary to supposed economy of metal, stiffness, etc.

At first thought it would seem that the thickness of the metal in the web, over and above the amount required to equalize the strength of the interlocked joint, should be eliminated, thereby getting less weight and consequent lower cost. If this were done, in some cases, it would weaken the strength of the pile as a beam, and also conflict with the author's second requirement, viz., stiffness to enable the piling to be driven without buckling.

Most demands for steel sheet-piling, at present, include lengths running from 30 to 50 ft., though engineers are now taking up propositions calling for still greater lengths, viz., 50 to 65 ft. Such increased lengths can now be manufactured and driven, thus meeting the requirements for strength, if sufficient bracing is used, spaced at proper horizontal planes so as to give safe spans for the pile as a continuous beam, supported on the different sets of waling timbers.

Experience in driving shows that it is not necessary for a steel sheet-pile to have the rigidity required in a column as commonly used in structural work. While it must be stiff enough to resist buckling under the first blows of the hammer, a slight spring under the hammer, within elastic limits, during the first penetration, is not detrimental.

The length of the pile as a column, above the point of penetration, rapidly decreases with each blow of the hammer, and, by the time the point of the pile has reached the hard, underlying formations through which the penetration is slower, the columnar length of the pile above the ground has been shortened, so that the tendency to spring under the blow has ceased.

Mr. Conkling.

The entering and interlocking of the joint with that of the adjacent pile already driven tends at once to give stiffness, practically fixing the lower end. Where a slotted, cast-steel driving cap is used, being held in position by grooves against the leaders of the driver, the top of the pile is also practically fixed. These fixed conditions at the top and bottom of the piece, assist in preventing a tendency to spring under the blow.

To illustrate this point, the writer has seen a 45-ft. pile which had been bent in unloading and handling on the work so that it had a middle ordinate of about 20 in. This pile was entered and interlocked with the one previously driven, and, by careful manipulation of the pile-driver, was driven to refusal, the bend in the piece being entirely taken out as it went down by the stiffness of the members of the joint of the adjacent pile previously driven, the final result in this case being a straight pile with perfect alignment in the wall. The vertical and horizontal support which one pile receives by interlocking with another already driven is far greater than is generally supposed.

Steel sheet-piling is often used in formations, the upper portions of which are composed of clay, muck, peat, quicksand, or other soft material. A single piece of piling weighing from 1 600 to 2 000 lb., by its weight alone, will enter such material, in some cases, many feet. If it were not for the friction in the interlocking joint, the weight of the hammer resting on the pile, taken in connection with the weight of the pile itself, would be sufficient, in most cases of this kind, to produce the first 10% of penetration.

On heavy work where the lengths of the pieces run above 50 ft., producing high tonnage and demanding economy of metal, it will pay to help out the pile in its first 50% of penetration by installing a series of movable wooden guides between the leaders of the driver, which latter will prevent the tendency to spring under the blow. It would seem economical, therefore, to require no more metal in the pile than is necessary to give it the required strength as a vertical beam after it is down, and just sufficient stiffness to get it down.

For these reasons the writer would place the requirement of stiffness as third instead of second, as given by Mr. Gifford, his third requirement, "Water-tightness," being given the second place.

Under this latter heading it would be possible to enter into a long discussion on the merits of the different types of interlocking joints, as affecting the final water-tight qualities of the wall. These different types naturally fall into two general classes, namely, rigid and flexible joints. Interlocking joints of the rigid type, when in normal position, have, in most cases, very small interior clearances between their members. The chance for the material which is displaced in driving to enter and seal the portions of the joints where the steel is not in actual contact is small as compared with joints of the flexible type.

Mr. Conkling.

These latter, with their large clearances or pockets, allow the displaced material to fill in these spaces and be compacted by the heads of adjacent joints when they are entered and driven, thus sealing the joints so that it is not necessary to use any artificial packing.

In driving a wall of steel sheet-piling, there are very few cases where stones in the form of large or small boulders, old logs, and other obstructions, are not struck. Sometimes these obstructions are sufficiently large, and, in the case of boulders, sufficiently hard, to prevent cutting through; especially is this condition to be met in deep work.

In such a case it is very desirable to be able, by the aid of the angularity of the flexible joint, to pass around the obstruction with the steel wall, driving to the same depth as previously done in the rear of the obstruction.

The flexible joint also allows the construction of circular or irregularly shaped pockets which are quite frequently desirable. By using joints of this type, these pockets can be built without the cost of bending the webs or additional fabrication.

In making closures in pockets, coffer-dams, or walls, the flexible joint allows distance to be gained or lost by the longitudinal displacement in the joint, or by deflecting slightly to the right or left.

These latter conditions also aid in bringing the piling back to vertical alignment, when, through meeting obstructions, or by careless driving, the top or bottom of the pile has been allowed to work ahead or back along the line of the wall. Where such inclination cannot be rectified, it may necessitate the manufacture of special tapered pieces in order to close the work.

The fixed joint, being somewhat rigid, reduces to some extent the first pressures on the waling timbers, where bracing is used in connection with the work. However, as soon as the excavation is carried to any appreciable depth, so that the pressures from the retained earth overcome the lateral stiffness of the joint, the work is thrown on the waling timbers, or, if no bracing is used, on the interlocked joints and the pile at or near the point of penetration.

It is probable that the perfect interlocking joint will finally be one which can be made flexible or rigid as desired, interlocking its members differently for each of these conditions.

Regarding the assumption that "the sections when interlocked act as a unit." By this the writer infers that the two parts of the joint are to be assumed as integral when interlocked. He does not believe this condition exists, except where the joints are grouted after the piles are driven. It is seldom that any form of steel sheet-piling is driven so that every piece making up the wall is in exact vertical alignment. Certain adjacent parts of the joints, therefore, may be in contact at any one point, and, further down in the same joint, similar points may be

Mr. Conkling. separated by a small space. Such latter spaces may be filled, at the time the pressure comes on the wall, with material displaced in driving, with silt carried in by water action, or the space may be left open. Also the small irregularities produced in fabrication and rolling tend to bring about similar conditions in the interlocked joints, all of which would prevent the two parts of the joint acting as a unit, at least in certain portions of the pile.

Therefore, if the section modulus is used as a relative comparison of the strength of the pile as a beam, it would seem better to use the section modulus of the full cross-section of one piece, or

$$\frac{\text{inertia of the section of one pile}}{\text{extreme fiber distance in same section}}.$$

The least radius of gyration, while the best comparison for theoretical stiffness as a column, should be considered in connection with the additional stiffness given to the piece by the interlock, density of material in which the pile is to be driven, and other conditions mentioned.

Economical water-tightness is certainly obtained, as explained by Mr. Gifford, by the non-requirement of auxiliary packing in the joints.

The methods generally used, to date, for specifications of steel sheet-piling, are not satisfactory, for the reasons which Mr. Gifford states.

It would seem safer to consider all pressures from saturated earth or quicksand as being hydrostatic. In cases where the wall is holding back such material, or dredged mud, it might also be better to consider the material as acting as a fluid, but having a weight of, say, 120 lb. per cu. ft., and to design the bracing to meet such conditions.

The writer would suggest that the working strength of the steel, in tension, might better be taken the same for temporary as for permanent work. Outside of deterioration of the steel in permanent work, which latter may not occur to any extent if the steel is protected, there would seem to be more liability for undeterminable pressures to be suddenly brought on the piling while in use in temporary constructions than in the more carefully planned and investigated permanent work.

Wood's formula, used by Mr. Gifford in his calculations of the strength of the pile as a beam, places the center of pressure nearer the center of the beam than is generally assumed by most authorities. This question, as well as similar ones presented in Mr. Meem's paper on the bracing of trenches and tunnels, is well worth the study and investigation of some of the expert mathematicians of the Society. There is no one line of work in which Civil Engineers are more often engaged than in building constructions to retain earth and water. If the long-used, old-time authorities are somewhat incorrect in their formulas, we cannot know it too soon for reasons of safety and economy.

With Mr. Gifford's permission, the writer would like to suggest the following additions to his specifications: Mr. Conkling.

Under "Unit Strains" he would add:

"The working stress on the extreme fiber, under any condition, shall not exceed 16 000 lb. per sq. in."

He would also add:

"*Strength of Interlocking Joint.*—The ultimate strength, in tension, of the interlocked joint of the piling, for any given unit of length, shall be not less than 25% of the ultimate strength, in tension, of the web of the pile for the same unit of length, this joint strength to be determined by a series of physical tests in a machine."

CHARLES H. HIGGINS, ASSOC. M. AM. SOC. C. E. (by letter).—In this able and valuable paper, as in other discussions on the same subject, the writer has seen no reference to the value of a relatively high section modulus in preventing the bending inward of the sheet-piling below the surface of the mud, inside the dam, during excavation. Mr. Higgins.

The writer may over-value this, but it is hard to believe that his experience with this difficulty is unique, for silt such as underlies the Lower Hudson, or some material similar to it, must frequently be met in the use of steel sheet-piling.

In the particular case the writer has in mind, the sheet-piling was driven to rock through some 30 or 40 ft. of silt, the water being 8 or 10 ft. deep.

For the benefit of those who are not familiar with the character of this silt at some little distance below the surface, it may be well to explain that this material, which is exposed at low tide, or is brought up by a dredge as soft mud, is so fine as to be held in suspension in water for a considerable time. In its natural condition, a few feet below the surface, it is a plastic solid, having a rather well-defined yield point and, when not exposed to water, standing readily with a vertical face of 8 or 10 ft., which would seem to measure what has been called its yield point; this increases inversely with the degree of saturation. A 4-in. cube, now before the writer, taken more than a year ago from the river bed, retains its corners and shows no signs of crumbling.

Sheet-piling, of one of the rolled sections mentioned by Mr. Gifford, was driven into this material, the water was pumped out, heavy bracing was placed, excavation was begun, and all went well. The bracing was placed immediately after removing about 6 ft. of silt, other bracing being placed every 3 ft. until a depth of some 15 ft. below the mud line, or from 20 to 25 ft. below the water line, was reached. Below this point difficulty was experienced in placing the braces, due to the sheet-piling bending inward below the line of excavation. This difficulty increased with the depth, and, although powerful

Mr. Higgins. hydraulic jacks were used, it became impossible to force the steel back to its original position; the area of the dam became more and more contracted, until at a depth of about 35 ft. below the water line, near the point where the rock was highest, the foot of the dam kicked in, and mud and water followed rapidly. The coffer-dam filled, and was only emptied and the excavation completed to rock by decking the entire structure and continuing the work under compressed air.

This sheet-piling had a radius of gyration of 0.8 and a section modulus of about 4.0 per ft. of width; it stood driving, even through old timber, behaved well with the bracing in place, and failed utterly through insufficient stiffness to withstand the unbalanced pressure in this plastic solid or cheese-like material below the line of excavation.

As far as the writer knows, this point has not yet been raised, although under certain conditions it would seem to be the determining feature in the choice of a steel sheet-piling, or even as to whether it might be advisable to use any form of an open coffer-dam. The writer calls attention to it in the hope of bringing forth some further information.

The huge coffer-dam of steel sheet-piling at Buffalo, mentioned as under construction, has recently been pumped out, and all interested in coffer-dams would be glad to learn what degree of success attended this operation.

Mr. Fowler. CHARLES EVAN FOWLER, M. AM. SOC. C. E. (by letter).—The use of metal sheet-piling is very limited in the Northwest, where the writer is located, and his opportunities for observing its use have been very infrequent.

When the writer discovered the paper by Mr. Borthwick, published in 1836 in the *Minutes of Proceedings* of the Institution of Civil Engineers, he was very much surprised that metal sheet-piling had not been used more extensively in the past. It is especially remarkable that, upon the extended use of wrought iron and steel in place of cast iron, metal sheet-piles were not used more generally, but the reason, doubtless, has been that, until of late years, timber for the construction of coffer-dams has been remarkably easy to obtain and has been of low cost, as compared with wrought iron and steel.

In a great many cases it is also true that for such temporary construction timber is better adapted than steel in any form, mainly owing to the fact that it is necessary to make connections of bracing and staging at various points, and these connections can be made more easily with timber than with metal. In view of this, it is doubtful whether metal will ever supersede entirely the use of timber for sheet-piling.

A great point in favor of metal, as compared with timber, is the fact that, with the various dove-tailed patterns in use, the piling can

be driven more closely and made more nearly water-tight than with any form of timber sheet-piling, even where a tongue and groove of dove-tailed form has been spiked to the timber, as this is apt to pull off in driving and leave bad leaks in the timber wall. Mr. Fowler.

With reference to the pressures on sheet-piling, the author has evidently overlooked the fact that in a very great number of cases the pressure is not that due to the water, but to that from semi-liquid or very soft mud, and may easily run up to 50% more than that of water.

The specifications for metal sheet-piling proposed by the author are certainly satisfactory as to the class of steel to be used, as there is no need of having it manufactured under any special specifications. In the matter of unit stresses for steel, it would be safe to use a fiber stress of 24 000 lb. per sq. in. in the temporary work, and 20 000 lb. for permanent work. The ratio of length to least radius of gyration is certainly high enough, but this is a matter which can be determined very largely from experience obtained by actual use of the various forms.

In the large majority of cases, the types shown in Group 1 would doubtless prove most satisfactory, owing to their greater sidewise stiffness and the better opportunities for making connections to them. There are, however, a great many cases, especially in shallow work, where the types shown in Groups 2 and 3 would be more satisfactory, on account of their lightness and less cost. Where the work is of any magnitude, it is always best to make a thorough investigation for each case, and adopt the form which is most satisfactory for the particular location.

J. C. MEEM, M. AM. SOC. C. E.—The speaker is a strong believer in interlocking steel sheet-piling, and, while he does not wish to appear as an advocate of any special type, he is glad to take this opportunity of speaking a word in favor of the more general use of this material. Mr. Meem.

Reinforced concrete piling is subject to the same fundamental disadvantages as those debited against wooden sheet-piling, in that it does not interlock and therefore may be forced apart by obstructions encountered in driving; and, further, that the corners cannot be made secure against an influx of sand or other material during excavating and pumping. Further, the speaker does not believe that the fear that harm may result from steel piling left in the ground is well founded. He bases this assumption on his belief that piling driven into the ground under water will not deteriorate after the outside heavy protective coating of rust has established itself, owing to the fact that there will be little or no further opportunity for oxidation, as long as the piling is not actually exposed to free water; and,

Mr. Meem. further than this, if the steel should rust out entirely, the space occupied by the rust or oxidized iron ought to be far in excess of that of the steel itself, leaving no opportunity for voids to occur.

The speaker believes that the use of interlocking steel piling in long lengths is too well established as an engineering factor to make it necessary to point out its advantages; but he is not so sure that it is as well-known a fact that this material can be used to great advantage at times in very short lengths. He has had occasion to use very short steel piling in connection with the underpinning of buildings and existing subways along the line of construction of new work, and the following is a brief description of the methods used:

The piers of the building or subway to be underpinned were first shored on temporary beams in the ordinary way; 6 by 6-ft. pits were then excavated and box-sheeted with ordinary 2-in. planking down to the level of the water, after which 4 by 4-ft. frames of interlocking steel piling of the "United States" type, usually in 6-ft. lengths, that is, sufficiently long to reach about 2 ft. below the sub-grade of the adjacent excavation, were set up and driven to full depth; after which the pits were mucked out to within a foot of their bottom and filled with concrete, the processes of excavation and concreting being carried on without unwatering the pile pit.

The result of this was that there was no influx of sand under the piling, as would have been the case had the pit been unwatered without having driven the piling to a much greater depth below excavation. As soon as the concrete in the pile pit had set sufficiently, the remainder of the pit was filled with concrete or bracing, and the work of underpinning was completed in the usual manner.

It is interesting to note that in many of these footings, which were nothing more than large square piles of from 1 to 2 ft. penetration, there was never any evidence of settlement, although they were at times subjected to a pressure of as much as 25 tons per sq. ft., with the adjacent material excavated to within at least 18 in. of the toe of the steel piling.

It is sometimes desired to use steel sheet-piling in very much longer lengths than can be handled conveniently owing to lack of head-room in building coffer-dams under streets and in congested places. In such instances it may be advisable to cut the piling into shorter lengths, driving the second and third sets as followers. Care should be taken, however, to break joints between the adjacent piles, and splice-plates should be tap-bolted into the ends of the abutting sections in order to keep one pile from crawling away from its follower while driving the adjacent member. Circular frames, 5 ft. in diameter, of steel piling in lengths of about 6 ft., have also been used to good advantage by the speaker as sumps for pumping out coffer-dams. Sumps of this shape can be driven continuously without the use of bracing, and are

very effective. Piling of the "United States" type in 6-in. widths has been used most extensively by the speaker in these operations, but piling of the "Lackawanna" type or any flexible type should be equally effective. Great care should be taken in the selection of steel piling to secure the type best suited to the conditions. Mr. Meem.

It has come to the speaker's notice that a coffer-dam designed to have its piling act as a vertical cantilever resulted in developing stresses similar to those found in a suspended chain stressed horizontally. This condition, fortunately, had been foreseen by the engineer in charge for the contractor, and he selected for use a piling which gave a high strength of cross-pull in the interlock. In relation to this matter Table 1 gives some figures which show a wide variation in cross-pull at the interlock, as found in the "Freistadt," the "United States" and other types. While the "Freistadt" is particularly effective as a beam, it has very little resistance to cross-pull in the interlock. The speaker is indebted to the Carnegie Steel Company for the figures in Table 1 relative to the "Freistadt" and "United States" types, and to the engineers of the Lackawanna Steel Company for the figures relating to the "Lackawanna" type. It is only fair to say, however, that a built-up section such as suggested by Mr. Skinner should also give good results if specially designed for effective resistance against cross-pull.

TABLE 1.

Type of piling.	Section of piling.	Resistance to cross-pull per linear inch at interlock.
"Freistadt" steel sheet-piling.	15-in. by 35-lb. channels, and 4-in. by ⅜-in. zees	1 500 lb. per lin. in.
	15-in. by 50-lb. channels, and 5-in. by ½-in. zees	2 250 " " " "
"United States" steel piling	12-in. by 40-lb.	12 000 " " " "
"Lackawanna" steel piling	12¾-in. by ½-in. web	9 970 " " " "
	12¾ in. by ⅝-in. web	9 970 " " " "

It would appear from the figures in Table 1 that very great advantage may be derived at times from designing coffer-dams so that the material in them will give thrust developing these stresses in the piling, thus saving a large amount of cross-bracing. These stresses can best be developed by driving the piling between supports with a "belly" or bow, making the versed sine of the latter as great as possible with reference to the horizontal distance.

MEMOIRS OF DECEASED MEMBERS.

NOTE.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

DANIEL DAWSON CAROTHERS, M. Am. Soc. C. E.*

DIED JANUARY 2D, 1909.

Daniel Dawson Carothers, the fifth son and seventh child of Rezin Dawson and Elizabeth Bain Carothers, was born at Cutler, Washington County, Ohio, on August 21st, 1860.

In early life, his father had been a millwright, but later he became a railroad contractor. He had a fine mechanical turn of mind, which his son inherited, and was noted for his determination and energy. His ancestors had come to America from Scotland, in Colonial days, and had settled in Huntington County, Pennsylvania. Mrs. Rezin Carothers was of English extraction, her ancestors having emigrated from England to Maryland, subsequently moving to Beaver County, Pennsylvania.

The early life of Daniel Dawson Carothers was passed on a farm, where he was required to do the "chores" and odd bits of work which fall to the lot of a country boy. This early experience tended to develop both mind and body, and the boy showed a strong liking for work with tools and for everything which pertained to machinery. His mother exerted a strong influence over his early life and intellectual development, giving him his primary schooling, as the neighboring country schools, in those days, were poor and the term of study very short. The last year or two of Mr. Carothers' school life was spent at Bartlett Academy; later, he went to the National Normal University at Lebanon, Ohio, where he took a special course in engineering, although he did not enter for a degree. During the last three years of Mr. Carothers' college life, he taught school during the winter months. Finally, in 1882, he began his labors in the field which he had chosen for his life work. At that time his father was connected with the Wheeling and Lake Erie Railroad, and the young civil engineer was given employment as a Rodman under his father, who early impressed upon him, as the primary necessity for success in business, the spirit which would "always obey orders."

As Rodman and Assistant Engineer, Mr. Carothers was with the Wheeling and Lake Erie Railroad during 1882. The next year he entered the service of the Columbus and Cincinnati Midland Railroad (now a part of the Baltimore and Ohio System), serving as Assistant

* Memoir prepared by Paul Didier, M. Am. Soc. C. E.

Engineer, Chief Engineer, and Train Master. He remained with this Company for seven years, or until 1890, when he transferred the field of his activity to the Baltimore and Ohio Railroad, with which road, or its subsidiary lines, he remained until his death.

When Mr. Carothers entered the service of the Baltimore and Ohio in 1890, it was as Engineer of Maintenance of Way of the Baltimore and Ohio Southwestern, which position he held until 1901. He then became Superintendent of the Chicago Division of the Baltimore and Ohio Railroad, with headquarters at Chicago; two years later, in 1903, he was made General Superintendent of the Baltimore and Ohio Southwestern, with headquarters at Cincinnati. In 1904 he was chosen Chief Engineer of the Baltimore and Ohio Railroad, with headquarters at Baltimore, which position he held at the time of his death.

Mr. Carothers was married on September 20th, 1888, to Miss Carrie Leland of Lewiston, Maine. They had one child, a son, who lived only a short time. Mr. and Mrs. Carothers resided in Baltimore for the past five years, and made many friends. Mr. Carothers was a Member of the Maryland, Merchants, Baltimore Yacht, Baltimore Country, and Engineers' Clubs of Baltimore; the St. Louis Railway Club; the Western Railway Club of Chicago; the American Railway Association; the National Geographic Society; the American Railway Engineering and Maintenance-of-Way Association, of which he was a Director and Chairman of the Rail Committee; a Member of the Committee on Conservation of National Resources, having attended the two conferences of the Governors of the States with the President of the United States, at Washington; the Ohio Society of New York; and the Madisonville Lodge of Masons.

Entering the service of the Baltimore and Ohio Railroad on the eve of a new era in railroading in America, while the many companies were beginning such marvelous strides in the way of development, Mr. Carothers played an important part in the upbuilding of one of the greatest railroad systems in the country. During the years he filled the many important positions to which he was chosen prior to 1904, he was instrumental in directing much of the energy then being devoted to railroad improvement.

Mr. Carothers was a man of pleasing personality, and typified, not only a success as an engineer, but a success as a man. As an engineer and railway official, he had initiative power, and his education and large experience, backed up by one of his most dominant characteristics—common sense—gave great strength to his views among other men of his Profession. As a man, his sterling qualities and manliness drew everyone to him. The men who worked in close touch with him were those who most appreciated and respected his opinions. Notwithstanding the position in the railway world which he had made for himself, those who knew him felt that this was but the

beginning of his true usefulness, and all experienced a deep sense of personal loss at his death.

"The world was better because he lived. A noble spirit has gone to rest."

Mr. Carothers was elected a Member of the American Society of Civil Engineers on April 4th, 1904.

RUTGER BLEECKER GREEN, M. Am. Soc. C. E.*

DIED DECEMBER 8TH, 1908.

Rutger Bleecker Green was born in Syracuse, New York, on October 8th, 1868, and was the second son of Andrew Heatley Green, a prominent attorney of that city, and Mary Miller Green. He prepared for college in the public schools of Syracuse, New York, and Morristown, New Jersey, and entered Cornell University in 1890. He was graduated with the Class of 1895, having lost one year because of sickness. As an expression of their appreciation of his ability and of their regard for him, his classmates, during his Senior year, bestowed upon him the highest honor within their power by electing him Chief Engineer of the survey made by the Senior and Junior classes at the northern end of Owasco Lake, New York.

Previous to entering college Mr. Green had had considerable practical experience, having been engaged as Chainman on the New York State Canals; Chainman and Instrumentman on the Richmond and Danville Railroad; Chainman on the Adirondack and St. Lawrence Railroad; and Assistant Engineer on the construction of the Syracuse Water-Works' pipe line from Skaneateles Lake, being in charge of a cross-section and construction survey party, on Sections 1 and 3, in 1893.

After graduation, Mr. Green was an Assistant on the editorial staff of *The Engineering Record* for nearly two years. He left *The Record* to accept the position of Leveler on the New York State Canals. In November, 1898, he accepted a position as Civil Engineer with the Detroit Works of the Solvay Process Company, in which position he remained until his death. He had charge of the land surveys of the 400-acre plant on the Detroit River; designed and constructed a yard system of 8 miles of tracks, with 75 turn-outs and an 8-stall locomotive house, 1 mile of concrete sewers, 2 miles of gas and water pipe for manufacturing and fire protection, including a yard system of 40 hydrants, and automatic sprinkler systems. He also supervised the construction of the foundations and superstructure of a 300-ft. railroad draw-bridge and coal-car tipple, and the enlargement of the

* Memoir prepared by M. S. MacDiarmid, Assoc. M. Am. Soc. C. E.

foundations of a 260-ft. chimney, also the foundations of various office and factory buildings, including lime-kilns, dynamo-house, ammonia tanks, concrete bulk-soda storehouse and office building.

Personally, Mr. Green's disposition was particularly loving and sympathetic, and all with whom he came in contact were pleased to call him friend.

Special mention should be made of his interest in religious and philanthropic affairs, as at the time of his death he was Treasurer of St. Mark's Episcopal Church at Delray, and a Trustee of D'Arcambal Home.

Too close application to his work, and the intense interest which he took in everything in which he engaged, resulted in a nervous breakdown. In June, 1907, Mr. Green took a trip to Europe for his health, but he was not permanently benefited.

A recurrence of the old trouble during the summer and fall of 1908 caused such a state of mental depression that, with the words, "I am worked out," he sought rest in death, on December 8th, 1908.

Mr. Green was elected a Junior of the American Society of Civil Engineers on May 5th, 1896; an Associate Member on October 5th, 1898; and a Member on September 6th, 1904. He was elected a Member of the Detroit Engineering Society on October 26th, 1900. He contributed valuable discussions to each of these Societies.

CHARLES MacRITCHIE, M. Am. Soc. C. E.*

DIED JANUARY 27TH, 1909.

Charles MacRitchie was born in Inverness, Scotland, in 1844. From 1860 to 1865, he served as an apprentice with G. E. Mackay, Land Surveyor and Civil Engineer, of Inverness, Scotland.

Mr. MacRitchie began his professional work in 1866, in the office of Edward Blythe, of Edinburgh, Scotland, being employed on the construction of the Dingwall and Skye Railway. Part of the time he acted as Resident Engineer on the western half of the road.

In 1869, Mr. MacRitchie came to the United States and, after a short stay in New York, went to Chicago, where he was employed in the private office of the late E. S. Chesbrough, M. Am. Soc. C. E. (who, at that time, was City Engineer), in making plans for a railway tunnel under the Detroit River, at Detroit, Michigan.

Under the direction of and in association with Mr. Chesbrough and the late Moses Lane, M. Am. Soc. C. E., Mr. MacRitchie made surveys and plans for the water supply of Pittsburg, Pennsylvania, and Long Island City, New York. He was also Principal Assistant Engineer

* Memoir prepared by John Nichol, M. Am. Soc. C. E.

on the construction of the water-works for Milwaukee, Wisconsin, and afterward served as Secretary of the Board of Trustees.

In 1875, Mr. MacRitchie entered into partnership with John Nichol, M. Am. Soc. C. E., under the firm name of MacRitchie and Nichol, Engineers and Contractors, which partnership continued until 1906. This firm prepared plans and contracts and carried out work, either for a complete water supply, or a part thereof, in many cities in the United States, which included some interesting submerged-pipe work in Galveston, Texas, Boston, Massachusetts, and St. Paul and Minneapolis, Minnesota, and also some railway-construction work.

From 1906 until his death, Mr. MacRitchie was in partnership with Mr. Benezette Williams, engaged in railway and other contracting work.

By his genial, kindly and generous disposition and straightforward honesty of purpose, Mr. MacRitchie made many friends wherever he went.

Mr. MacRitchie was elected an Associate of the American Society of Civil Engineers on October 2d, 1872, and a Member on April 5th, 1876.

CHARLES TARBELL DUDLEY, Assoc. M. Am. Soc. C. E.*

DIED SEPTEMBER 30TH, 1908.

Charles Tarbell Dudley was born in Richmond, Indiana, on April 6th, 1878. He received his technical training at Sheffield Scientific School of Yale University and at the Colorado State School of Mines. He was graduated from the former with the degree of Ph. B. in 1900, and from the latter with the degree of E. M. in 1902.

While a deep student and an investigator along original lines, his memory is particularly lasting at these two schools because of his tireless efforts in the up-building of clean, wholesome, college athletics. Though small, Mr. Dudley was an athlete of the wiry, determined type, and Harvard and Princeton still remember him as a most dangerous half-back, centerfielder, and batsman. Determination and a high moral standard were his chief characteristics, and these, coupled with a cheerful, lovable disposition, endeared him to all his associates.

Before taking up his post-graduate work at the Colorado State School of Mines, Mr. Dudley secured employment as Assistant Chemist at the Midvale Steel Works. In six months his ability was recognized to such an extent that he was put in charge of the Department of Metallography and Experimental Chemistry, and in another six months he had invented a new process for the treatment of large, nickel-steel

* Memoir prepared by H. L. Haehl and C. E. Gilman, Assoc. Members, Am. Soc. C. E.

forgings, which nearly eliminated the imperfections so frequent before that time.

After his post-graduate course, Mr. Dudley went to California and acted as Superintendent of various mining properties until September, 1903, when he accepted employment with the Engineering Department of the Bay Cities Water Company, which corporation was at that time designing a new water supply for the City of San Francisco.

In 1905 Mr. Dudley resided in the Santa Cruz Mountains, California, and designed the trestles and bridges for the Santa Cruz Portland Cement Company's railroad. The great California earthquake of 1906 interrupted this work, and, seeing the opportunity for designing and structural engineers in the rebuilding of San Francisco, Mr. Dudley moved to that city. He was engaged in the design of some of the largest of San Francisco's new buildings when he was seized with an attack of typhoid fever, and for six weeks he fought for his life with his characteristic determination. Before he had regained his strength he moved to Allston, Massachusetts, and began again the practice of his profession. His strong constitution, in its weakened condition, could not stand this added strain, and a general breakdown resulted, against which his determination and grit were unavailing. Mr. Dudley died on September 30th, 1908. His wife, Sarah Emery Dudley, whom he married in Boston, in 1905, and two children, Sarah and Wade, survive him.

To those who knew him his death was a keen blow; to his friends, it was a great loss—the loss of that great possession, a wholesome, manly man and a true friend.

Mr. Dudley was elected a Junior of the American Society of Civil Engineers on September 6th, 1904, and was transferred to the grade of Associate Member on October 3d, 1906.

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AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

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CONCRETE PILES.

By Howard J. Cole, M. Am. Soc. C. E.

To be Presented September 1st, 1909.

The rapid decrease in the available timber supply—and the consequent increase in the price of all kinds of wood during the past thirty years—has served as a stimulus for the invention of methods for the conservation of this valuable building material. Investigations have proceeded along two distinct lines; first, to find a means whereby the serviceable life of the wood could be prolonged; second, to find a material which could be used as a substitute for wood. The first was accomplished by the discovery of the creosote treatment, which has more than doubled the actual term of service of wood in those industries in which it has been used. The second problem has been solved to a large extent by the development of the uses of concrete, both plain and reinforced, which is now replacing not only wood, but also steel, masonry, and other materials which have been standards in construction work for years.

The construction of concrete piles is one among the many uses to which concrete has been applied successfully as a substitute for wood. The first patents, in the United States, were taken out within the last decade. In that short period concrete piles have grown rapidly in

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

favor, and are now replacing timber piles for the support of heavy structures in wet or marshy situations. One concrete pile company states that during the past year its business has increased several hundred per cent., as compared with the previous year, although the period has been one of unusual financial depression.

It is a well-known fact that timber piles will last practically forever when constantly submerged in water, and there are on record many authentic instances where they have been found intact after a service of several hundred years. Probably the most conspicuous example that has been brought to the attention of the world in recent years is the foundation of the Campanile of St. Mark's, in Venice, built in 900 A. D., the superstructure of which fell in 1902 on account of weakness of the masonry due to other causes, and not from any fault in the pile foundation. The piles were found to be in such good preservation that they are being used to support the reconstructed tower. These piles were installed under favorable conditions, and were constantly submerged, but where timber piles are subjected alternately to the action of air and of water they soon decay.

The lowering of the ground-water level around a structure supported on timber piles will eventually cause damage to it and possibly its destruction. Many instances of this kind have been caused by the construction of the numerous tunnels and subways in New York City, as the level of the ground-water in their neighborhood has been reduced so much that the stability of pile foundations near them has been menaced. One very prominent building, constructed on timber piles only a few years ago, in a location which naturally required a foundation of that type, is endangered from this cause and, in the near future, steps will have to be taken to strengthen the foundation.

Where hard driving has been necessary in order to place timber piles in position, engineers have entertained a doubt as to the final condition of the pile and its real ability to support the load imposed on it, owing to the fact that many piles driven under these conditions and removed later have been found to be damaged seriously by being either telescoped or buckled. Recently, during the construction of a pneumatic foundation in New York City, a line of caissons was sunk along the side of the lot directly against the adjacent building, which rested on a timber pile foundation. A number of the piles had been driven so that they projected over into the other lot, and these were

removed as the caissons were sunk. Some of these piles were buckled and crimped into accordion-like folds, and had no sustaining power whatever; how many more are in a similar condition is a matter of conjecture. Timber piles are injured more frequently in driving than is generally supposed; but comparatively few are ever removed, and the integrity of the foundation is taken as a matter of fact. In the construction of caisson foundations for sky-scrapers in New York City, it has been the custom in many instances to drive timber piles to form a temporary support for the derrick traveler. These piles remain in place until all the material has been excavated, and are then removed; and, although they have been selected carefully and driven by expert workmen, a large percentage is found to be badly damaged.

In the construction of piers, docks, and similar structures, if timber piles are installed in waters frequented by marine borers—the teredo or the limnoria—their life is very short; in badly infested waters an untreated pile may not last eight months, while a creosoted pile may last for years, though its period of service is somewhat doubtful, as piles similarly treated and placed do not last the same length of time.

The concrete pile has none of the disadvantages of the timber pile, as it cannot be affected by the reduction of the ground-water level, or by the attacks of marine borers, nor is it injured by hard driving. It can be driven into ground impenetrable to the timber pile, can be made of the dimensions and form desired, and be proportioned to the load that it is to support; at the same time, every pile can be made to comply fully with the specifications. Those who have inspected timber piles realize how difficult it is to obtain pieces which will fill the demands of the specifications in all particulars, as to size of butt, size of point, straightness, etc., especially when lengths of more than 50 ft. are required. The concrete pile has a greater carrying capacity than the timber pile, individual piles having been tested with pig-iron loadings of 45 tons and more without settlement, and piers of five piles have supported satisfactorily a test load of 300 tons, or an average of 60 tons per pile. In practice, however, it is not desirable to load them to more than 25 tons per pile, unless reinforced to provide for the excess loading. In good practice, timber piles of a comparable size would not be loaded to more than one-half of this

FIG. 1.—TIMBER PILES DAMAGED IN DRIVING: EXPOSED BY EXCAVATION.

FIG. 2.—SUPERSTRUCTURE FOR OIL TANK COLUMNS SUPPORTED ON TYPE A OF THE MADE-IN-PLACE CLASS.

amount, therefore the carrying capacity of the concrete pile is double that of the timber pile.

The method adopted in the United States to demonstrate the carrying capacity of a concrete pile has been to make an actual loading test of some of the piles by placing a platform on a pile, loading it with pig iron to double the guaranteed load, and, by a series of accurate levels, keeping watch for any settlement that may occur.

Some attempts have been made to calculate the supporting power of concrete piles by the formulas generally used with timber piles, but it is quite evident that none of these formulas will apply in cases where the concrete pile is jetted into place. When it is driven into position, however, a formula like "Ritter's," which takes into account the weight of the pile, can be used with good results. The "Ritter" formula is:

$$W = \left(\frac{h}{e}\right)\frac{Q^2}{Q+q} + Q + q,$$

in which W = The resistance to further penetration,
Q = The weight of the hammer, in pounds,
q = The weight of the pile, in pounds,
h = The height of the fall, in inches,
e = The penetration, in inches, by the last blow.

The concrete pile costs more per linear foot than the timber pile, but, in many installations, some items of construction, necessary with the timber pile, are eliminated entirely or reduced materially by the use of the concrete pile, so that the final result shows a considerable saving in time and money with the latter. For instance, where timber piles are used and the ground-water is several feet below the surface, it is customary to excavate to the water level and support the sides of the excavation with sheathing before driving the piles; then, after the piles are in place, they are cut off at the proper elevation, the pier or wall is built to the required height, and the excavation is backfilled. This proceeding is simplified greatly with the concrete pile foundation, where the pile is driven from the surface of the ground, without any preliminary excavation, and only requires a capping extending below the frost line to prepare it for the superstructure. Under these conditions, the concrete pile foundation saves on the items of excavation and depth of pier while it eliminates entirely the

items of sheathing, pumping, and back-filling, and at the same time accomplishes the result in a much shorter period of time.

The ideal location for a concrete pile is in filled areas where the water level is at considerable distance below the surface and the ground is of such a nature that it is necessary to carry the footings to a firm underlying stratum; here, the concrete pile can compete with the usual concrete or masonry pier, as, under these conditions, it will save most of the items mentioned in the example just given. In the construction of foundations for a large warehouse at Pittsburg, Pa., on the site of an old fill, from 20 to 30 ft. deep, where it was necessary to carry the foundations to a firm stratum, bids were received on two methods of construction: one being to carry the loads to the proper depth by concrete piers in open excavated pits, and the other to use concrete piles. The lowest time limit set by the open-pier method was 18 months; the work was actually done by the concrete-pile method in 76 days, thereby effecting a saving of more than 15 months in time alone, as well as a considerable amount of money.

Comparisons of the relative cost of concrete and timber pile foundations in fair-sized installations are not common, but an instance is cited in detail of a foundation at Annapolis, Md., which shows a saving in money of more than 50% on the cost of a timber pile foundation:

2 193 timber piles at $9.50	$20 833.50	
4 542 cu. yd. excavation at $0.40	1 816.80	
3 250 cu. yd. concrete at $8.00	26 000.00	
5 222 lb. I-beams at $0.04	208.88	
Shoring and pumping	4 000.00	
		$52 859.18
855 concrete piles at $20.00	$17 100.00	
1 038 cu. yd. excavation at $0.40	415.20	
986 cu. yd. concrete at $8.00	7 888.00	
		25 403.20
A saving of		$27 455.98

The saving in material was:

1 338 piles,

3 504 cu. yd. excavation,

2 264 cu. yd. concrete,
5 222 lb. I-beams, and
all shoring and pumping.

Although the concrete pile is cheaper under favorable conditions, the timber pile will be used for many years, especially in places where the ground-water level is near the surface and the building to be supported is so light and cheap as not to warrant an expensive foundation.

There are many kinds of concrete piles on the market. Most of these are patented and are the varieties in general use; they can be divided into two general classes, according to their method of construction: First, those made in place, where the pile hole is made by a mandrel and the concrete is dropped directly into the position it is to occupy as a concrete pile and sets in place; and second, those moulded or cast and allowed to harden thoroughly before being placed in position. For piles of this class the pile hole is not prepared in advance, but the pile is driven or jetted into place in a manner similar to that used with timber piles.

These piles all differ from each other in shape, being either straight or tapering, and in cross-section being square, polygonal or circular. These variations in form have led to considerable discussion as to the merits of different combinations, but it is the intention in this paper to mention in an impartial manner only the relative bearing powers of straight and tapering piles, without going into the comparative merits of the others, leaving that element to be brought out in the discussion, with the details of construction, cost figures, etc.

In the first class to be described there are two representative patented types. Both of these were first patented and used in the United States, and are distinctly American; consequently, installations of this class have been more numerous than of the second or moulded class. One of these, which for purposes of identification will be called Pile "A," is formed by driving to refusal a hollow steel pipe or form, of the proper length, $\frac{3}{4}$ in. in thickness of shell, 16 in. outside diameter, and of the same diameter throughout its entire length. This hollow form is driven by an extra heavily built pile-driver, which, in addition to the usual equipment, has a strong pulling device attached to the leads for the purpose of withdrawing the form from the ground after driving. The lower end of the form has a special jaw or shoe which

is closed during driving, in order to prevent the entrance of the surrounding materials into the form; in certain soils the "Alligator Jaw" point is used. This jaw is attached securely to the form by cable hinges, and is used over and over again. It is composed of two symmetrical wedge-shaped parts which are kept closed during driving by the pressure of the soil. Generally, a detachable cast-iron point is used, but this is more expensive than the "Jaw," as it is left permanently in place, and one must be supplied for each pile. The upper end of the form is reinforced with a strong band of steel shrunk in place and secured firmly to the form with countersunk rivets; this steel band also serves as a shoulder for the clamps which are attached to the pulling device. A steel driving head, sliding in the leads, with a circular projection on its lower face to fit into the top of the form, is lowered on the form, and not only serves to hold the top of the form firmly in place during driving, but also receives the impact of the hammer, from which it is protected by a rope mat.

When the form has been driven to refusal, the driving head is hooked to the hammer by a wire attachment, and both are hauled to the top of the leads and chocked there during concreting operations. A charge of concrete—about 5 cu. ft.—is dropped into the form, after which the entire form is raised about a foot by the pulling device; then a heavy iron rammer is dropped on the mass and by its impact opens the jaw and forces the concrete out into the space made by the end of the form. This procedure is repeated until the form has been entirely withdrawn from the ground and the space it occupied is completely filled with concrete, which hardens in place and becomes a concrete pile.

This method of filling is varied to meet varying conditions; occasionally, after the jaw has been opened, a sufficient quantity of concrete to make a complete pile is placed in the form and the form is steadily withdrawn; this plan, under certain sub-surface conditions, gives a better shaped pile, because, under the first method, the jaws have a tendency to close partly on account of the excess of pressure at each stop of the form, thus making a pile of irregular section; this is avoided by giving a regular and steady movement to the form. The concrete is made quite wet, of a 1:2:4 mixture, using ¾-in. stone, and, when placed in the form in mass, is heavier than the surrounding soil. Thus it is more than able to balance the pressure, and the soil has

FIG. 1.—TYPE A, MADE-IN-PLACE CLASS: PILES EXPOSED FOR INSPECTION

FIG. 2.—MOULDED PILES, SIXTH STREET VIADUCT, KANSAS CITY.

no tendency to cut off any section of the pile, as the excess pressure serves to force the concrete into the soil.

In very hard driving, heavy forms, even with a ¾-in. shell, have been buckled at the top, and one can judge from this what the condition of a timber pile would have been under similar conditions. In one case the form was driven into the lower stratum so firmly that the pulling device could not withdraw it, so it was filled with concrete and left permanently in place.

It is claimed for piles of this type that the fresh concrete, which comes into direct contact with the surrounding material, enters into all its irregularities and unites with it, thus forming a rough exterior surface, which gives the pile a large amount of frictional resistance; and, further, as practically all the resistance to driving is confined to the point, the greater part of the compression takes place where the pile needs it most in order to transfer its load as a column.

In locations where the strata penetrated by the pile are in a fluid or semi-fluid condition, and have not enough consistency to serve as a mould to retain the concrete in shape, piles of this type could not be used without material modification. In one case of this nature, after the driving form had passed through the soft soil and had been driven to refusal into the firm underlying stratum, the initial charge of concrete was inserted and the point was forced open; then an auxiliary form of thin No. 20 metal, of slightly smaller diameter than the driving form, was dropped into place, and the required reinforcing was adjusted. Then the auxiliary form was completely filled with concrete, and the driving form was withdrawn, leaving the thin shell in position to act as a mould for the concrete. The foundations for a public utility building on Staten Island, N. Y., were constructed in this manner, the resulting pile being one which had its point embedded in the bearing stratum 8 ft. and was 16 in. in diameter in this portion, while the column part of the pile was 12 in. in diameter, and was enclosed in a shell of No. 20 metal. These piles stood successfully the test loading.

The length of piles of the ordinary type driven by this system is between 20 and 25 ft., but piles 48 ft. long have been used. The largest installation of this type is in the foundation of the Pittsburg Terminal Warehouse, Pittsburg, Pa., where about 6 000 piles, from 30 to 45 ft. long, were placed.

The other leading type of this made-in-place class, Pile "B", differs considerably from the one just described. It is tapering in shape, and each pile has a steel shell, to prevent the earthy material from mixing with the concrete, this shell being left permanently in position. A heavy pile-driver, usually on a turn-table frame for facility of operation, is used to drive a tapering collapsible form or core into the ground to make the pile hole; this core weighs about 3 tons, and is built in three longitudinal sections with provision for horizontal movement that permits the sections to be expanded or collapsed. Before beginning operations the core is suspended in the leads, with the point about 8 ft. above the ground to allow clearance for the sections of casing to be slipped on. The casing is made from sheets of No. 20 metal, 8 ft. long, which are formed into the required shape, the edges being bent to connect together and retain the shape of a truncated cone. These are of various sizes, to fit the tapering core, and are slipped on in regular order with a considerable overlap to each section, enough sections being used to cover the core completely to the length required. Occasionally, the joint between the sections is covered with an extra section to make a tighter joint. When this is done, a pressed-steel boot or thimble is forced on the point of the core overlapping the last section of casing. The core is now ready for driving. It is slung back into the leads, lowered carefully to the proper location on the ground, and is then driven to refusal by a steam hammer weighing about 3 tons. When the core has been driven to the desired depth, it is collapsed and withdrawn, leaving the steel casing in the ground as a lining for the pile hole; this is immediately filled with 1:2:4 concrete, using ¾-in. stone or gravel, mixed rather wet, which sets and forms the concrete pile. The concrete cannot be placed until the core has been withdrawn. In practice, it has been found that these protecting shells, being sectional, are not tight, but permit the entrance of water and quicksand which has to be siphoned out before the concrete is placed. When installed in locations where the shells are subjected to great hydrostatic pressure, the casing is sometimes collapsed and must be re-driven. In very hard driving in boulder formation, the core has been injured so severely as to be put out of commission and has had to be replaced by another.

It is claimed for this pile that its strong tapering shape, 20 in. at the butt and 8 in. at the point, distributes the load throughout the

entire length of the pile and compresses the soil to such an extent that the full bearing of the upper layers is fully utilized, and also, that the metal casing prevents the surrounding soil from mixing with and weakening the concrete. The Hospital Buildings of the United States Immigration Commission, at Ellis Island, New York Harbor, were installed by this method.

These two varieties of the made-in-place class have been more popular than any others in the United States in building foundations. They are reinforced occasionally with rods which are put in position as the concrete is deposited, or single rods may be churned into place through the wet concrete. The rods are left with their ends projecting above the concrete 1 ft. or more, in order to assist in bonding the capping to the piles. This made-in-place class is not adapted for use in docks, piers, wharfs, etc., without very material modifications of the typical pile described.

The installation of these piles has not always been a success; there have been failures, due to various causes, from which much has been learned. In one case all the subsoil conditions were favorable, the driving was satisfactory, and everything indicated a very successful piece of work; but when the physical test was applied the piles failed to support the load. On investigation it was found that the fault lay with the cement. One of the standard brands had been used, but it had been accepted without test, and proved worthless. In other cases difficult sub-surface conditions were met, each of which required some change in operations; but, at the present time, an unsuccessful installation is rare when all the conditions of the problem are carefully studied and investigated.

No single type of concrete pile will fit all locations without some change to suit the site, and, before determining what particular pile shall be used, a very careful survey of the subterranean strata should be made by taking test borings in several parts of the proposed site. Good wash borings will generally furnish this information, but core borings are more reliable though somewhat more expensive. A few years ago it was customary to drive test piles, in order to determine what length of pile to specify in the contract, but this plan was usually rather expensive, unless a pile-driver was on the site. In one instance four timber test piles cost $200 each, on account of the expense of moving the pile-driver to the site. Test borings should show the

various strata, their thickness, etc., and the elevation of the ground-water; the strata will be of different thicknesses and elevations in different parts of the site, and the borings should provide the information necessary to determine the length of pile, the type of pile to be used, and on which stratum to place the load. Neglect of these precautions is responsible for some of the settlements which have occurred in pile foundations, both timber and concrete. One instance may be cited where a 10-ft. fill had been placed on a swampy piece of ground, the bottom of which was 40 ft. below. To have reached this hard bottom, the piles should have been 40 ft. long, but a tapering pile of about two-thirds that length was used, and, for its support, relied on the filled layer and the spread of the concrete capping, and thus made what might be termed a floating foundation. This foundation settled slightly at first, and continued to go down somewhat each year until the total settlement is now more than 3 in.

One advantage of made-in-place piles over moulded piles is that, as they harden in the place where they are to be used, a small foundation could be completed before the moulded piles would be ready to drive, thus effecting a small saving in time.

In the construction of the foundations of a garage at Albany, N. Y., an unpatented type of this class was used, which serves to show one of its possible variations. The original surface of the site was a stiff clay and had been covered to the depth of 15 ft. by clay from adjoining excavations; thus it was necessary to carry some of the load to the original surface. This was accomplished by excavating holes, 10 in. in diameter, through the overlying fill to the original surface, using a screw post-hole digger, and then filling the holes with fresh concrete, the clay filling being stiff enough to retain its position during the operation. The piles were spaced at 3-ft. centers, two staggered rows being placed under the walls, while the pier loads were supported by clusters similarly spaced. The total length of piles was 4 779 ft.; the labor of digging the holes cost 20 cents per lin. ft., and the concrete $7 per cu. yd. in place, or about 13 cents per lin. ft., making a total of 33 cents per lin. ft. of completed pile. In pile work there is usually quite a large charge for plant installation and dismantling, but in this instance this was reduced to a minimum, which accounts for the low unit cost. No compression of the pile hole was made, but the supporting power of the pile was assisted materially by the capping, which acted as a spread footing.

FIG. 1.—METHOD OF MOULDING AND LOWERING COMPLETED PILES, ATLANTIC CITY BOARDWALK.

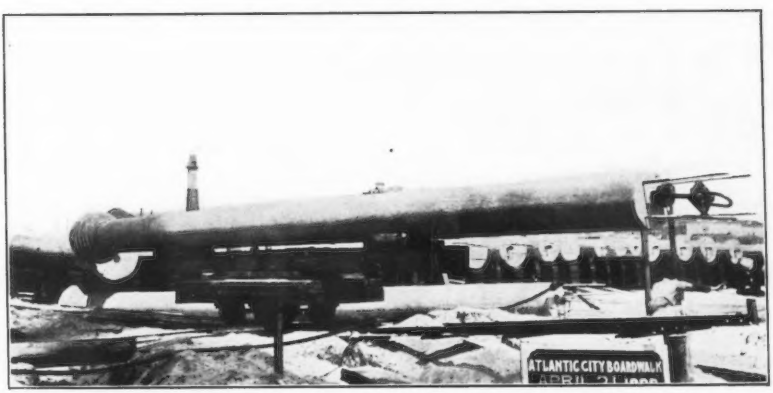

FIG. 2.—VERTICALLY MOULDED PILE, ATLANTIC CITY BOARDWALK.

Another type of this class has been used in the clay soils around Boston, Mass., where the clay was overlaid by some 20 ft. of sand and silt. Through this an 8-in. casing was forced into the clay and the materials were washed out. Then a chambering device was used to expand the footing of the pile by cutting away the clay, which was also removed by washing. In this way the area of the footing of the pile was made 16 times as great as that of the pile itself—a very effective method of obtaining a spread footing at considerable depth at a moderate cost. This type of pile has been patented.

Just previous to the construction of the pneumatic foundations of the Trinity-United States Realty Buildings, at Broadway, Cedar Street, and Trinity Place, New York City, the foundations of three columns of the Sixth Avenue Elevated Railroad, located on the curb line of the Trinity Place front, were carried to hardpan, 75 ft. below the curb, by using steel-encased concrete piles. This was done in order to avoid any possible settlement of the railroad structure during the sinking of the caissons, which were to be carried to bed-rock. The original piers of these columns were founded on the sand at about ground-water level, 12 ft. below the curb. Below this footing elevation there was a thick stratum of quicksand extending to the hardpan, the latter resting directly on the bed-rock. The transverse girder supported by the first column was shored up, and the column and its pier were removed and then restored completely before any work was done on the next column. Over the close-sheeted excavation made for the removal of the old pier there was used a short pile-driver which could be operated under the overhanging girder, and this was assisted by a water jet, to force a 12-in. standard, black, wrought-iron pipe, in random-length sections, to the hardpan. When the first section had been worked down to the bottom of the sheeted excavation, another section was joined to it with outside couplings, the threads on the ends of the pipes being cut long enough to allow the ends of the sections to butt properly. The lower end of the bottom section was protected in most cases by an open driving shoe, which made it easier to sink the pile shell than with a cone-shaped, closed point, as it allowed the free use of the jet to soften up the material. The upper end of each section was protected from the direct impact of the hammer by a driving head which screwed into its coupling and butted on the pipe. This driving head had a slotted opening in its side through which the jet was worked during the operation of the hammer.

Most of the material from the inside of the pipe was removed by the jetting process; any that remained was taken out by an ordinary sand pump. There was no difficulty in getting out all the material after the pipe had reached the hardpan, the depth being checked by steel tape measurements, and the interior of the pipe inspected by the aid of a lantern. The pipe was then filled with 1:2:4 concrete. Five piles were thus placed for each pier, one near each corner, and the fifth in their midst directly beneath the center of the column. The piles extended 5 ft. into the mass of concrete which formed the new pier, and was 8 ft. square at the base and stepped in to meet the column base at the curb level.

A similar pile has been used in the underpinning of buildings in New York City. It is placed directly beneath the wall by cutting in the latter a vertical opening, from the starting elevation and through the foundation courses, of sufficient width and depth to leave ample working space around the pipe. The sections are forced into the soil by hydraulic jacks assisted with water jets, using the remainder of the wall for reaction purposes.

These piles are of various diameters, from 8 in. to 3 ft., in proportion to the loads to be carried, and the spaces are cleaned out in a manner similar to that just described, except that the largest sizes, in which there is space enough for a man to work, are cleaned by hand. The spaces are then filled with concrete under air pressure. When the pile is completed the load of the building is transferred to it by masonry and wedging stones. It is the most effective system of underpinning in use to-day, as it gives a positive assurance of the safety of the building, and at the same time does not occupy any space outside the wall, thus the contractor has a lot free from all obstructions.

Some of the tall buildings in New York City are supported on foundations of a patented type composed of steel-encased concrete piles, heavily reinforced throughout, and driven to place in a manner somewhat similar to that used in sinking other encased piles, except that a pneumatic hammer is used in connection with the water jet. The shell of this pile is generally $\frac{3}{8}$ in. in thickness and 12 in. in exterior diameter, the sections being joined by inside couplings. The reinforcing rods are inserted in sections, which are fastened together by an effective mechanical device, and form a continuous rod. The

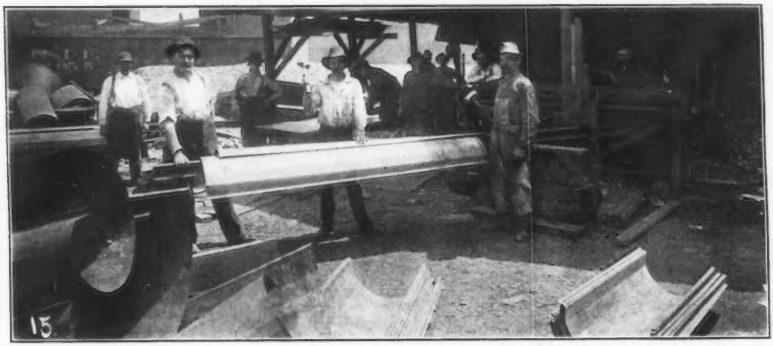

FIG. 1.—TYPE B, MADE-IN-PLACE CLASS; METHOD OF MAKING CASING.

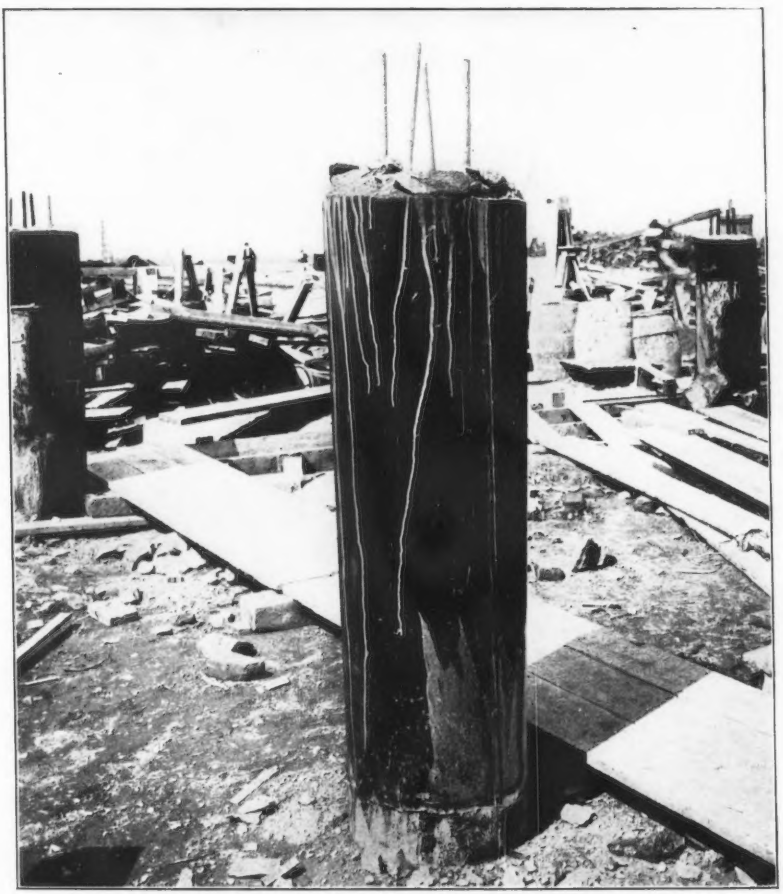

FIG. 2.—TYPE B, MADE-IN-PLACE CLASS; PILE EXTENDED ABOVE GROUND TO RECEIVE CAPPING.

earthy material in the shell is withdrawn, and is replaced by concrete, in the manner heretofore described. On account of the large area of steel in its cross-section, the Building Department of New York City has given this pile a much higher bearing value than any other, namely: 350 lb. per sq. in. on the concrete in compression and 4 200 lb. per sq. in. on the steel. The life of the steel in the exterior shell is somewhat doubtful, as it is subjected not only to the oxidizing influence of the ground-water, but also to the corrosive action of gas in the soil, and to stray electric currents; therefore it should not be considered as a permanent element in the strength of the pile.

The cost per linear foot of the made-in-place class varies with the type of pile, amount of reinforcing, the size of the contract, the accessibility of the work, and the sub-surface conditions. As no two pieces of work are alike, it is difficult to give more than a general idea of the cost. In the representative types of this class the cost will vary from 75 cents to $1.50 per linear foot of pile, depending on the foregoing conditions; the steel-encased varieties are more expensive, as the casing alone costs more than the above-mentioned price.

Concrete piles of this class should always be reinforced in order to provide for all cross-strains due to sub-surface conditions or those which may take place during construction, and while the pile is still green, particularly when the length of the pile is more than 15 ft. Piles of this class are better adapted to lengths of less than 30 ft., while, for greater lengths, a moulded type is more desirable, as there is doubt as to the homogeneousness of the concrete when it is dropped 30 ft. and cannot be thoroughly spaded in place.

In the second class of concrete piles—the moulded type—the patented varieties, like those of the first class, are more generally known than the unpatented. They also differ from each other in shape, cross-section, and method of reinforcement. The moulded pile was first developed in Europe, and is used extensively there, and also in the United States, for piers, docks, and similar structures, for which it is specially adapted. It is also being used in the foundations of buildings and other structures where subsoil conditions are favorable, and especially where long piles are desired. Piles of this class, more than 60 ft. in length, have been successfully driven, pulled out, and re-driven several times without showing any signs of weakness. All moulded piles are reinforced to stand the rough handling to which

they are subjected in transportation from the moulding yard to the site and in being placed in the leads, as they are sometimes hauled up by a line attached to the butt of the pile, instead of by a bridle which would lessen the cross-strain. The driving strains are very great, and it is thought by many that a pile with reinforcement should not be subjected to the impact of a hammer; but many piles have been driven in this way, and later pulled up and re-driven, without showing any damage by the hammer. In one instance an attempt was made to destroy a pile by hard driving, but the only effect was to mash or broom the butt of the pile for about 2 ft., the remainder being uninjured.

Most of the moulded piles are driven into place; but, when conditions are favorable, they are jetted, and in some varieties special provision is made for jetting by inserting a 2-in. pipe in the center of the pile. In city work it is seldom practicable to use the jet, on account of danger to the foundations of adjoining property. When

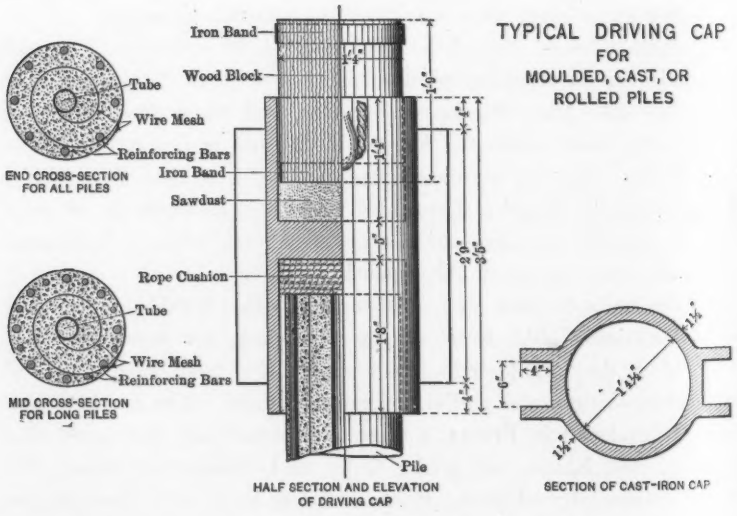

FIG. 1.

driven into place, some provision is made for cushioning the blow of the hammer. This is usually done by the insertion of a false pile of wood, acting on a sawdust or sand cushion, between the hammer

FIG. 1.—TYPE B, MADE-IN PLACE CLASS: FORM ENCASED IN METAL, READY TO DRIVE.

FIG. 2.—TYPE S, MOULDED CLASS: HAULING 61-FT. PILE INTO LEADS.

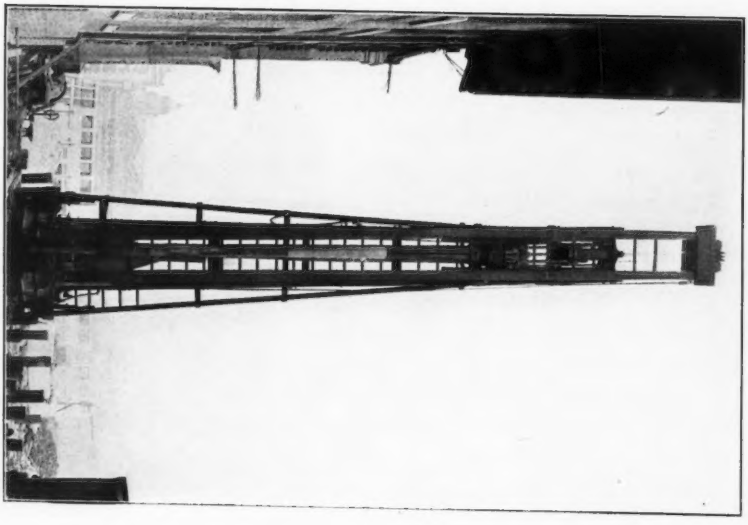

FIG. 1.—TYPE B, MADE-IN PLACE CLASS; FORM ENCASED IN METAL, READY TO DRIVE.

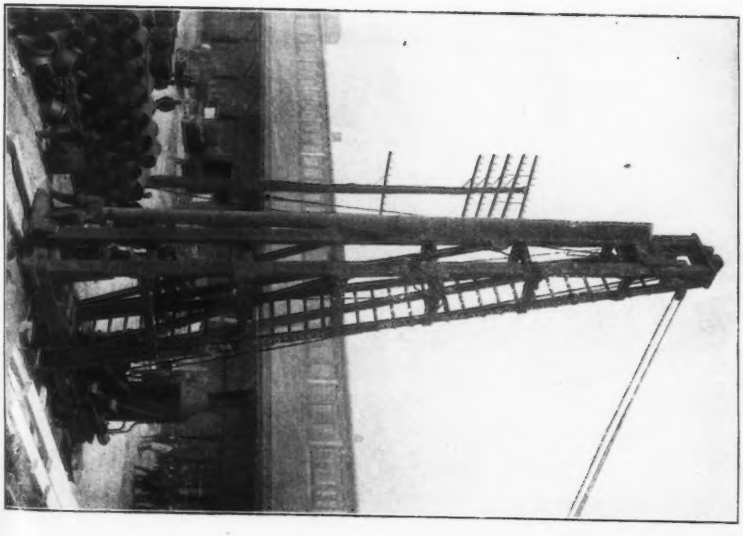

FIG. 2.—TYPE 3, MOULDED CLASS; HAULING 61-FT. PILE INTO LEADS.

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and the head of the pile, the whole driving head being held in position on the pile by an encircling metal cylinder held securely in place.

Piles of this class are moulded either horizontally or vertically, one variety being rolled instead of moulded or cast. The vertical system of moulding is the more expensive in time and labor, but, when thoroughly done, it produces a good pile. It is possible to effect considerable saving in forms alone by the horizontal system, as the upper portion or lid of the form for square or polygonal cross-sections can be omitted entirely, while the side pieces can be removed after 24 hours and used again, thus leaving only one portion—the bottom piece—tied up by each pile. In the vertical method all the sides of the form for the pile must remain in place until the pile is strong enough to stand alone. On a large installation quite a sum of money can thus be saved on forms, and, naturally, the horizontal method is the more popular.

One of the best known types of this class, and among the first to be invented and practically applied, is a straight pile of square cross-section with the corners slightly chamfered. It is reinforced with ¾-in. rods, one near each corner of the square, and connected with each other at intervals of 12 in. by diagonal and straight wire ties, the lower end of the pile being protected with a pyramidal metal shoe firmly attached to the reinforcement. Provision for jetting is made, in the lower end only, by the insertion of a piece of wrought-iron pipe, about 3 ft. long bent in such a manner as to extend from the center of the point to the outside of the pile just above the shoe, where an outside connection can be made and then removed, thus making it possible to save a considerable length of pipe and at the same time make a more solid pile. This type has been used in all kinds of foundation work, both in the United States and abroad, particularly in structures along water fronts, bridges, buildings, etc.

Another well-known type of the moulded pile is tapering in shape, 16 in. in diameter at the butt and 11 in. at the point. It is octagonal in cross-section, each of the faces of the octagon containing a semi-circular longitudinal corrugation for almost its entire length. It is reinforced throughout with Clinton woven-wire cloth, and a central opening, for jetting purposes, is moulded in the concrete by the use of a tapering collapsible mould, thus obtaining an opening for the water jet at small expense. After the pile has been placed, this open-

ing can be filled with grout and fine concrete, thus making a solid pile. A driving-head, somewhat similar to that previously described, is used with this pile when it is driven into place by impact. It is claimed for this pile that, as the corrugations increase its superficial area, its bearing capacity is increased correspondingly, and the corrugations assist materially in the escape of the water, making it more easily put in position.

A third type, circular in cross-section, is formed in an entirely different manner, in that it is not moulded or cast but is rolled mechanically into shape. A sheet of ordinary black wire fabric, of No. 16 gauge, with ½-in. mesh, and of the length required to make the pile, is attached to the rod which serves as the winding core and later as a part of the reinforcement of the pile, by small rivets driven into holes slightly smaller. The other edge of the sheet is attached to a movable platform which is pulled toward the core as it is wound up. On this sheet is placed the other reinforcement, and the whole is covered with a layer of fine concrete. The mass is then wound up like a jelly roll, and fastened with wire ties every 6 in., and more frequently where needed. In some of the later piles the method of fastening the outer edge of the wire cloth has been changed by omitting the wire ties and substituting a rivet with its end upset so that it is slightly larger in diameter than the mesh of the cloth. These rivets are inserted in the outer edge of the cloth with this upset end upward, and, as the pile is rolled up, they are forced into the meshes of the fabric in the body of the pile, thus tying or rather buttoning it together. The platform on which the pile is moulded is inclined, so that, without any lifting, the completed pile is rolled therefrom to the car which is to carry it to the seasoning yard. The platform of the car is also inclined, and, when the car has reached the yard, the pile is rolled to its place on skids. By this method no large plant, except the rolling device, is required to handle the pile during its formative stage. When these piles are to be driven into place, a driving head is used, and when jetted, a wrought-iron jetting pipe forms the winding core. Piles of this kind, 61 ft. long, have been driven, pulled out for examination, found to be intact, and re-driven. Like other piles of this class, they have stood very severe treatment. In one instance an attempt was made to destroy a pile by over-driving; later the pile was pulled up and the only apparent injury was the

FIG. 1.—TYPE B, MADE-IN-PLACE CLASS: FOUNDATION READY FOR CAPPING.

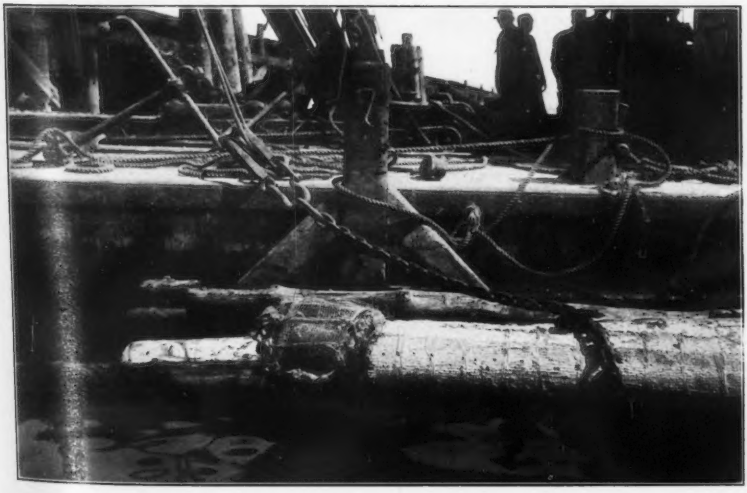

FIG. 2.—TYPE 3, MOULDED CLASS: EXAMPLE OF PILE DRIVEN TO REFUSAL SEVERAL TIMES.

mashing or brooming of the upper 2 ft. This type has been used in the foundations of some of the stations of the Brighton Beach Railroad, in Brooklyn, N. Y., in filled ground. It has also been used in other places throughout the United States.

At Kansas City, Mo., solid octagonal piles (unpatented) were used in the foundations of the Sixth Street Viaduct. These piles were moulded horizontally, and, when given sufficient time to harden, were successfully placed. The contract was let in the fall, and as it was found to be impossible to mould enough piles to complete the work, it became necessary to use some creosoted piles.

Moulded concrete piles were used during 1908 in the construction of a boardwalk, 50 ft. wide, at Long Beach, Long Island. These piles were moulded horizontally, and were 11 in. square at the butt and 8 in. square at the point, with the edges chamfered; they were reinforced with a 1-in. bar in the center. The piles were 25 ft. long and were jetted 15 ft. into the sand by the use of a twin outside jetting apparatus which worked very successfully.

In the construction of a pier at Atlantic City, a few years ago, there was used a moulded concrete pile with a bulbous end of much larger diameter than the body of the pile. These piles were moulded on end and jetted into place. The pile was 25 in. in diameter, and the bulb at its greatest diameter was 3 ft. 6 in. This pile was reinforced with eight $\frac{3}{4}$-in. bars symmetrically placed around a $2\frac{1}{2}$-in. jet pipe in the center.

During 1908 a moulded pile of the same general style was used in the construction of a boardwalk at Atlantic City, N. J.

In 1906 the Atlantic and Birmingham Railroad built two piers at Brunswick, Ga., on moulded piles somewhat like the pile of this class which was described first. The piles were 16 in. square, and were straight, except that the lower 10 ft. tapered to 8 in. square at the point, and the edges were chamfered. The top of each pile, for 22 in., was 10 by 16 in. to allow of a bolted connection with the wooden girders of the superstructure. They were reinforced with four $1\frac{3}{8}$-in. rods held in position at 12-in. intervals with wire clips, and fastened to the jet pipe at the point. These rods were extended to within 3 ft. of the top of the pile, the reinforcement in this part being woven-wire fabric. About 1 500 of these piles, from 32 to 50 ft. long, were installed. One would naturally expect that timber piles would have

been used, as Brunswick is the shipping port for yellow pine and its products, but it was not practicable to use timber on account of the marine borers. The supports of the timber docks now in use require constant repairs, and the timber pile does not last long enough to pay for its installation, even though it may be very cheap. On the other hand, these new concrete piles make a permanent pier and will never require replacing.

In a power-house foundation at Bridgeport, Conn., a composite timber and concrete pile was used. The site was on the banks of a tidal stream where the muck deposit was very deep, and this pile was used in order to save the expense of excavation and concrete footings. The timber pile was driven about flush with the surface of the ground, then a hollow, steel pipe follower, 12 in. in diameter, was used to drive it below water level. The follower was then filled with concrete and withdrawn, leaving the concrete in place on the butt of the pile. A mechanical bond between the timber and the concrete was obtained by the use of long metal dowels which were driven into the butt of the timber pile before the follower was placed in position.

Moulded concrete piles, of rectangular cross-section, and having a male and female joint on two edges, have been used as sheet-piles in many cases during the past few years. One type is 6 by 14 in. in cross-section, with a reinforcing rod near each corner, the rods being held in position by wire clips at regular intervals. These sheet-piles are installed in a manner similar to that heretofore described, either by jetting or driving, or a combination of both, as may be desirable.

There are in use to-day many other varieties of concrete piles which differ in some respect from those described, either in the reinforcing, the shape, or the method of installation, but their general characteristics will place them in one of these classes.

The different shapes and sections of the various concrete piles have led to considerable discussion between their respective advocates as to the relative bearing power of each pile, most of the contention referring to the shape of the pile and whether the tapering or the straight shape (in which the pile is of uniform cross-section throughout) has the greater supporting value.

The tapering pile depends for its bearing power more on its wedge shape and the skin friction thus developed than on its efficiency as a column, while the straight pile relies on the skin friction developed

FIG. 1.—TYPE 3, MOULDED CLASS: MACHINE FOR ROLLING THE PILE.

FIG. 2.—TYPE 3, MOULDED CLASS: PILE 14 IN. DIAMETER; 50 FT. LONG; WEIGHT, 3½ TONS. HAULED 4 MILES TO SITE OF WORK.

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by its exterior surface, and also on its strength as a column. The amount of skin friction developed by any pile depends, not only on the character of its surface, but also on the area of that surface. Considering for the present only the area, and assuming two piles of equal length and having butts of the same size, one tapering to 6 in. at the point while the other is straight throughout, it will be found that the straight pile has a larger surface area on which to rely for skin friction, and has a larger bearing surface at the point. If these piles are each 14 in. at the butt and 20 ft. long, the tapering pile will have a surface area of 52.36 sq. ft. as compared with 73.30 sq. ft. for the straight pile; while the point area of the tapering pile will be 0.196 sq. ft. and that of the straight pile 1.069 sq. ft., the latter having almost 5½ times as much area available to transmit its load as a column, about 1.4 times as much superficial area, and 1.7 times as much volume, which also represents its displacement.

The relative compressions may be seen by Table 1.

TABLE 1.

Type of Pile.	Diameter, in Inches.		Circumference, in Feet.		Cross-Section Area, in Square Feet.		Surface Area, in Square Feet.	Volume, in Cubic Feet.
	Butt.	Point.	Butt.	Point.	Butt.	Point.		
Tapering.	14	6	3.665	1.571	1.069	0.196	52.36	12.64
Straight..	14	14	3.665	3.665	1.069	1.069	73.30	21.42

Comparing the two piles again, for the purpose of ascertaining the relative frictional values of their upper and lower parts, it will be found that the tapering pile has a much larger proportion of its surface area in the upper than in the lower zone, which indicates that it must depend on the upper layers of the ground for the greater part of its supporting power, while the straight pile has its surface area uniformly distributed and can utilize all the strata through which it passes.

It is apparent from Table 2 that 32.5% of the surface of the tapering pile is in its upper quarter while only 17.5% is in the lower quarter. The Building Department of New York City allows 15 tons per sq. ft. on rock and 10 tons per sq. ft. on firm gravel or clay, when foundations

are carried down through earth by piers of stone, brick, or concrete; on this basis, assuming a load of 20 tons on each pile, and that each has been driven to the firm gravel, so that it can act as a column, the tapering pile, with its 0.196 sq. ft. of area at the point can transmit as a column only 1.96 tons, leaving the remainder of the load, 18.04 tons, to be supported by skin-friction, and about one-third of this would have to be supported by the upper stratum of the ground. The straight pile, having 1.069 sq. ft. of area at the point, can sustain as a column 10.69 tons, leaving 9.31 tons to be carried by skin friction, and about one-quarter of this, or 2⅓ tons, would be supported by the upper stratum of the ground.

TABLE 2.

	Circumferences, in Feet.		Superficial Areas of Zones, in Square Feet.		Percentage of Zone Areas to Entire Pile.	
	Tapering	Straight.	Tapering.	Straight.	Tapering.	Straight.
Butt	3.665	3.665	17.020	18.325	32.5%	25%
One-quarter	3.142	3.665	14.400	18.325	27.5%	25%
One-half	2.618	3.665	11.780	18.325	22 5%	25%
Three-quarters	2.094	3.665	9.160	18.325	17.5%	25%
Point	1.571	3.665				
			52.36	73.30	100.0%	100%

The basic reason for the use of piles for foundations is that the surface ground at the site has not sufficient stability to support the load; therefore piles are used as a means by which to transmit it to a lower and more substantial stratum. Structures supported on tapering piles, in districts subject to floods and the scour of currents, are likely to settle seriously if erosion takes place, and, under these conditions, the piles should be driven to a firm bearing stratum and not rely on the upper one to support the load.

The supporting power of a pile due to frictional resistance depends largely on whether the surface of the pile is smooth or rough, thus a rough-surfaced concrete pile will have more sustaining power, due to that element, than a pile of the same size with a smooth metallic or wooden surface. The supporting power due to the same factor

also depends on the material in which it is driven, which may be anything from a fluid muck to a solid gravel or clay; it is possible to have any of these features in combination with those above mentioned, therefore it is important to study carefully all the conditions of each problem before deciding which type of pile to adopt.

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AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

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COPYRIGHT IN DRAWINGS OF A TECHNICAL CHARACTER.*

By D. A. Usina, Assoc. Am. Soc. C. E.

The copyright law enacted March 4th, 1909, to go into effect July 1st, 1909, contains provisions which may be of great importance to engineers, in that it provides specifically for copyright on "Drawings (or plastic works) of a scientific or technical character," and in that the securing of the copyright is made an extremely simple matter.

Prior to the passing of this law there has been no certain protection against the pirating of designs for engineering structures. Where a design though new has involved only the skill of the engineer, it has been unpatentable, and unless the work could be classed under the fine arts there has been no reasonable hope of protection by copyright. In many cases designs involving a high degree of engineering skill and gotten up at great expense have been copied by others with no moral justification whatever. National, State and city governments and private parties have asked for bids to be accompanied by original designs and have appropriated the design of one bidder and let the contract to another—a species of piracy distinguished legally but not morally from the piracy of works of literature or the fine arts.

*This paper will not be presented at any meeting, but written communications on the subject are invited for publication with it in *Transactions*.

It is hoped that the new law will put a material check on such practices by reserving to the proprietor of such drawings the sole right of printing or copying them. It is doubted if the law can be invoked to prevent the building of the structures shown in such drawings, with the possible exception of works which might be classed in the fine arts. The limitations which can be put upon the use of the drawings, however, should afford a substantial degree of protection, and the ease with which copyright can be secured makes it undoubtedly worth trying for whatever protection can be secured.

The language of the Act relative to technical drawings is as follows:

"That the application for registration shall specify to which of the following classes the work in which copyright is claimed belongs: * * * (*i*) Drawings or plastic works of a scientific or technical character."

Taken in connection with the remainder of the law, there is little or no doubt that this language was intended to render such drawings copyrightable. The view of the Register of Copyrights is:

"that the language of the Act would authorize the deposit and registration in this office for copyright protection of drawings of a scientific or technical character such as under the present law would not be included in the term 'drawing,' which seems to be confined to artistic drawings. Examples of such works would be drawings for machinery, for engineering construction, for architectural works, etc. While these drawings have to some extent been registered in the Copyright Office heretofore, it has always been a question whether the law authorized the registration and whether copyright protection would follow."

Protection is extended to "all the copyrightable component parts of the work," such for example as the drawings relating to any complete and mentally segregable part of the whole structure.

Copyright is secured by publication with the required notice affixed to each copy; or, where "not reproduced for sale," by depositing a photograph or other reproduction with the Register of Copyrights. But engineering drawings are usually offered for sale (as part of the bidder's work), and, in such cases, the copyright is secured by affixing a notice of copyright; and after thus securing the copyright there must be promptly deposited in the Copyright Office two complete copies.

Several permissible forms of notice are provided. It is sufficient if the word "Copyright" be applied to each sheet of drawings, together

with the name of the proprietor and the date. It is customary now to apply the name and date to drawings; and the addition of the word "Copyright" is the simplest matter in the world. For what it is worth (and apparently it will sometimes be worth much) the word "Copyright" should be added to all original drawings and copies thereof.

In the case of drawings of especial value it is advisable also to deposit copies promptly with the Register of Copyrights and to secure a certificate of such deposit, so as to obtain the apparent sanction of the Government, and to indicate such fact on the face of the drawings. For drawings of less value the depositing of copies may be delayed until it is thought necessary to threaten or bring suit for infringement. The securing of copyright is probably effected by the mere notice. The deposit of copies is an additional formality which, though required to be made "promptly," and though a necessary preliminary step before a suit for infringement can be brought, does not seem necessary to the securing of the right.

Copyrighted works may be sold or furnished to others with any reasonable limitation upon their use. For example, a bidder may furnish an original design with a notice of copyright and a special notice that the copyright and the copies submitted constitute parts of the bid and remain the property of the bidder in case his bid is not accepted, and that the design is not to be copied or submitted for bids from others.

An interesting feature of the new law is the apparent sanctioning of actions against the United States and its officers; but much weight cannot be placed on this construction, in view of the broad prohibition in the general law against actions "sounding in tort" against the United States. Congress has provided no remedy for wrongs committed by the National Government, except for violations of contract, express or implied. The provision of the new law bearing on this subject relates specifically to the recovery of costs in suits for infringement and states that in such suits,

"except when brought by or against the U. S. or any officer thereof, full costs shall be allowed, and the court may award to the prevailing party a reasonable attorney's fee."

It is unnecessary to consider the question here at length, but it may be stated generally that after a tort is committed by the United States there is no adequate remedy, but an injunction may be secured

against a repetition of it; and perhaps, if one has sufficient notice of an intention to infringe the copyright, such infringement can be enjoined in the first instance. Possibly the framers of the law had such injunctive proceedings in view when they framed the above section exempting the United States and its officers from the payment of costs.

The Act provides also that "any person who willfully and for profit" infringes a copyright may be punished by a fine and imprisonment; and an infringer who had received such a special notice as is above referred to would be infringing willfully and for profit, and would hesitate to run the danger of fine and imprisonment.

Under the old copyright law there were only a few cases in which it was attempted to secure a copyright upon a purely technical drawing. It was necessary to enter the drawings in the office of the Librarian of Congress, and the Register of Copyrights permitted such entry only under protest and was of the opinion that such drawings were not copyrightable. The validity of the copyright was generally doubted. The new law, which specifies such works particularly as one of the classes in which copyright may be claimed, and permits the securing of the copyright by applying to the drawings a notice that copyright is claimed, is a material advance toward the protection of a kind of intellectual property of far greater dignity and usefulness than circus posters and similar matters, to which the previous law had extended protection. To quote the language of the Constitution upon which is based the power of Congress to make a copyright law, technical drawings do more "to promote the progress of science and useful arts" than many of the classes of work heretofore provided for.

The practical value of the new law to engineers cannot be predicted. We can only hope. It cannot be used to protect functional equivalents of the thing represented by the drawings. That is the purpose of the patent law, and protection thereunder runs for seventeen years only, while copyrights may last fifty-six years. It is quite possible for the courts to construe the new law so narrowly as to destroy its value. But it will probably be construed so that it will protect the engineer against the copying by others of the substance or of any material part of his original designs.

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PAPERS AND DISCUSSIONS

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HYDRO-ELECTRIC POWER IN CANADA.

By Cecil B. Smith, M. Am. Soc. C. E.

To be Presented September 1st, 1909.

Introductory.

To view justly the progress which has been made in Canada, in developing water-powers, up to the present time, certain premises must be established, climatic conditions realized, the trend of population, habits, and occupations of the people pointed out, and, above all, the peculiarly favorable topography of the country described.

The population of Canada is familiarly spoken of as agricultural in character, but this is only relatively true, as large and important industries flourish throughout the Eastern Provinces, and, indeed, it is painfully evident that even in this young and sparsely populated country the increase in population is almost entirely urban in character, except on the great wheat plains of the Middle West.

Industries are based on labor, raw materials, and power, and only in the last named would the ultimate conditions be unfavorable to continued progress if coal were the only source of power to be relied on, because Canada is very peculiarly situated as regards its coal areas.

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

Four areas are at present known, as follows:

The Nova Scotian, which is on the extreme Atlantic seaboard, and produces a good bituminous coal, available as far westward as Montreal;

The Souris, which is in Southwestern Manitoba, and produces a low-grade lignite of limited area and value;

The Alberta, which covers a very large area extending from the United States boundary to north of Edmonton and from the plains well up into the Rocky Mountains, the quality ranging from lignite to anthracite, and, as regards both quantity and quality, probably one of the most valuable coal deposits in the world; and

The Vancouver, which is very extensively distributed over the island and up some of the mainland valleys, chiefly developed for export to the Pacific Coast cities of the United States.

Thus it appears that coal is available on the extreme west and east, but for 2 000 miles, covering much of the most populous and fertile center of the country, it is necessary to import coal from the United States for heat and power.

From a hydraulic point of view, however, the country has been peculiarly favored, as a glance at the map will show.

The Hudson Sea is deep set into the heart of the country, and the height of land between the waters flowing to this sea and to the St. Lawrence drainage system is from 1 200 to 1 500 miles long, and, which is of more importance, it is only from 1 000 to 2 000 ft. above the sea, is not of a mountainous character, but is a perfect network of lakes, swamps, rock, and forest, and affords a good reservoir and regulator for hundreds of large rivers, which, as they reach the parent St. Lawrence, descend over precipitous Laurentian rapids, admirable as sources of hydraulic power. This condition obtains all over Canada as far west as Eastern Manitoba, and, as the rainfall averages from 25 to 40 in. per year, with no marked tendency to drought, and as the long winters store up so much moisture that the low-water period often merges into the period of autumn rains, the pinch of low water is not in many cases a serious menace, except in March of severe winters. On the great western plains the topography and climate both militate against hydraulic values, as will be dealt with later.

In British Columbia there are two diverse regions: the eastern and central portions have a low rainfall, and the glaciers feed the

rivers throughout the summer, so that the low-water period is in the late summer and autumn.

Along the Pacific Coast there is an excessive rainfall of from 100 to 150 in. per year, but it is distributed unequally throughout the seasons, and only by making use of storage lakes can extreme low-water conditions be avoided.

Until twenty years ago, hydraulic power often formed the basis of growing industries and determined frequently the location of the centers of population, but the larger centers of trade were developed by rail and water transportation and in many cases were quite distant from such sources of power.

With the advent of electric power transmission, however, these cities offered a favorable market for the sale of light and power from distant sources, and private companies, and in some cases municipalities, have invested large sums in hydro-electric constructions and transmission systems to serve the adjoining districts.

While much has been accomplished, much remains to be done, and the discussion of present accomplishments which is herein given is intended to mark a mile-stone on the road.

Of the many great rivers of Canada, the Mackenzie, Nelson, and St. Lawrence are by far the greatest, having drainage areas of 660 000, 370 000, and 565 000 sq. miles, respectively; but, outside of these, there are three districts worthy of notice from a hydraulic standpoint, namely, the Atlantic Coast, the Pacific Coast, and that great array of rivers which, in addition to the Nelson, flow into the Hudson Sea.

Atlantic Coast.

On the Atlantic seaboard the configuration of the Provinces of Prince Edward Island and Nova Scotia does not provide large drainage areas, and, also, in the presence of extensive coal measures and coal mining, the incentive to hydro-electric development has been lacking; indeed, a recent installation of steam-electric equipment having a capacity of 500 kw. at the pit mouth, from which electric energy is transmitted at 11 000 volts to Amherst, N. S. (distant 6.5 miles), may be but the pioneer of much more extensive installations of a similar character in other parts of America.

Farther westward in New Brunswick, there exists the condition more usual in Canada, namely, rivers having large drainage areas,

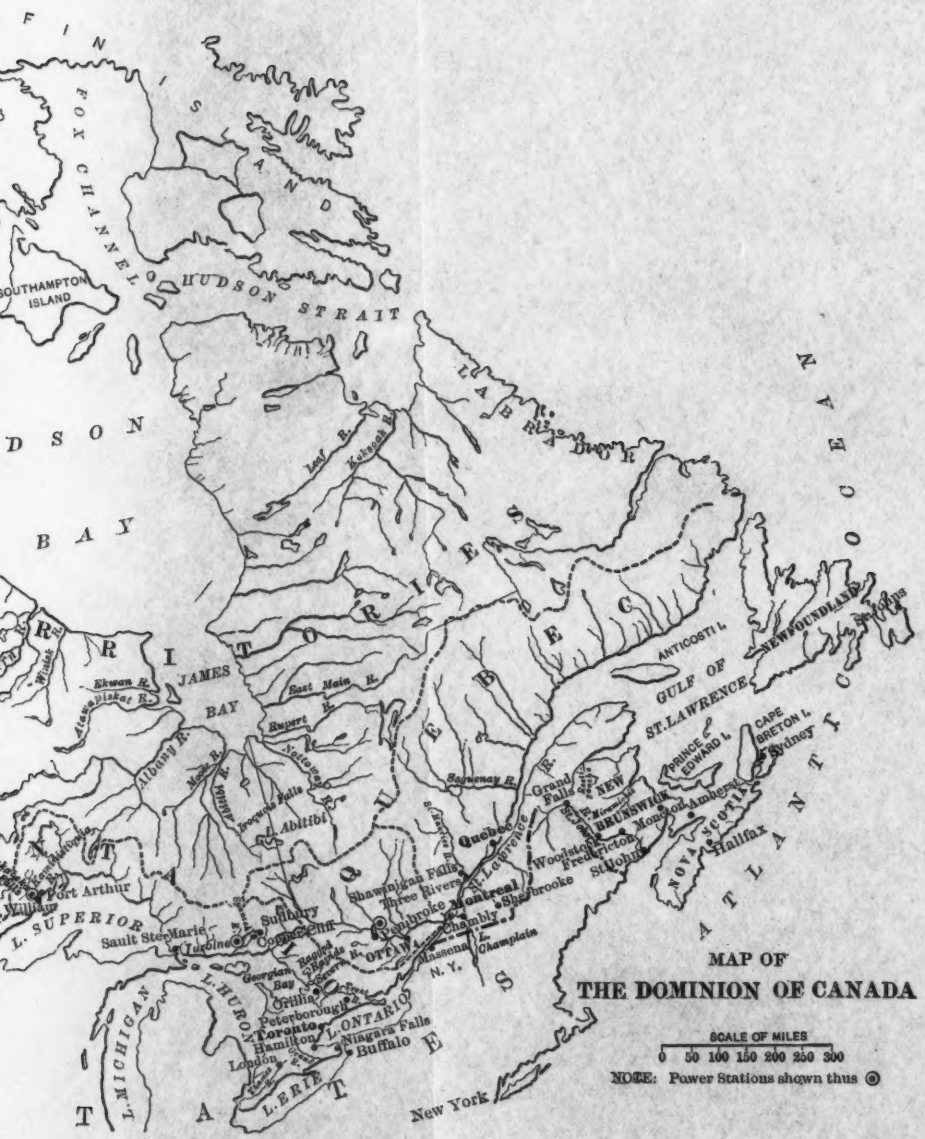
MAP OF
THE DOMINION OF CA

SCALE OF MILES
0 50 100 150 200 250 200

NOTE: Power Stations shown th

VICTORIA
ISLAND
B A F F I N
FOX CHANNEL
SOUTHAMPTON
ISLAND
Dubawent
L.
H U D S O N
B A Y
Athabaska
Thlondeer
Churchill R.
Nelson R.
Saskatchewan
Lake
Winnipegosis
Winnipeg
MANITOBA
L.
Manitoba
Regina
Assiniboine R.
Winnipeg
Lake of the Woods
Port Arthur
Port William
Fort Frances
Rainy R.
Red R.
L. SUPERIOR
Sault
L. MICHIGAN

PLATE XIX.
PAPERS, AM. SOC. C. E.
MAY, 1909.
SMITH ON
HYDRO-ELECTRIC POWER IN CANADA.



MAP OF
THE DOMINION OF CANADA

SCALE OF MILES
0 50 100 150 200 250 300

NOTE: Power Stations shown thus ◉

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and having their sources in swamp, forest, lake, and muskeg, in which snow and frost linger late in April, or even until May, and feed these rivers throughout the dry months of August and September; indeed, it is usual in Canada to find the extreme low-water period in March, before the frost has ceased to hold the moisture from flowing. The Miramichi, Restigouche, and St. John are great rivers, and extensive hydraulic constructions may be expected for grinding pulp alone, in addition to serving near-by towns and cities by transmitted electricity.

A most interesting and extensive development has been commenced at Grand Falls, N. B., by the Grand Falls Power Company, where the St. John River, with its sources in hundreds of lakes of Northern Maine, falls abruptly 115 ft. over a slate barrier.

For these hydro-electric works the type of construction which has been selected is that of vertical, 10 000-h.p., 3-phase, 25-cycle, 11 000-volt units, operating under a head of 130 ft., and the designs, under which contracts for the construction of the works are about to be let, provide for a hydraulic capacity of 80 000 h.p., with a present machinery installation of 40 000 h.p., which will generate current for ferro-manganese smelting, pulp grinding, and for transmission to the cities of Woodstock (70 miles), Fredericton (125 miles), and St. John (165 miles). An interesting feature of the design is that the generators are to be placed in the wheel-pit immediately above the turbines, with a provision for the proper ventilation of this subterranean chamber as well as careful water-proofing to ensure satisfactory electrical conditions. The tunnel tail-race will be placed entirely below the level of the tail-race water, and the draft-tubes from each unit, therefore, will be brought into the top of the tunnel from one side of the wheel-pit; this, it will be noted, is a variant on the methods adopted by the Toronto and Niagara Power Company in its works at Niagara Falls, Ont.

Pacific Coast.

On the Pacific Coast, the hydraulic conditions are very different from those in other parts of Canada, for the great mountain ranges are not good reservoirs, and, although the glaciers feed the rivers until late in the summer, yet at other seasons, owing to the light rainfalls east of the Coast Range, the rivers become very much reduced

in volume; thus the excessive floods of the Fraser River, for instance, make it unsuitable for hydraulic development for power purposes.

Near the Coast, however, the rainfall is excessive, and the existence of small lakes has facilitated the construction of several interesting developments, serving Vancouver and the adjoining territory. As instances, those of the Stave Lake Power Company and the Vancouver Power Company may be mentioned.

The works of the Stave Lake Power Company, now under construction, are 35 miles east of Vancouver, on Stave River, where a head of 90 ft. is to be utilized. The drainage area, estimated at 360 sq. miles, provides a large volume of water, owing to the excessive rainfall, which varies from 100 to 150 in. per year. The flow in the river is estimated to vary from 2 000 to 50 000 cu. ft. per sec., making the river regulation somewhat difficult.

The equipment, as at present designed, will consist of five units, each of 5 000 h.p., and it is considered that the electrical output from this installation has a good prospective market in Vancouver and Westminster to which the current is to be transmitted at an ultimate voltage of 60 000.

The Vancouver Power Company, Limited, has a most interesting development at Butzin Lake, 18 miles northward from Vancouver. This has been made possible by diverting the water of Coquitlam River at Lake Coquitlam into Butzin Lake by a 2½-mile tunnel having a section of 73 sq. ft.; thus the water required for operating the plant is conveyed from the Coquitlam drainage area, estimated at 100 sq. miles, the run-off varying from 300 to 5 000 cu. ft. per sec. under an excessive rainfall of 150 in. per year. At Butzin Lake the water is taken from the head-works by steel pipes to a power-house, 1 800 ft. distant, and located at sea level, 400 ft. lower.

The generating equipment is typical of the Pacific Coast, being originally composed of four units, each of 3 000 h.p.; two Pelton wheels for each unit drive a 3 000-h.p., 2 300-volt generator at 200 rev. per min., in addition to which there is also a recent installation of a 10 000-h.p. generator driven by four Pelton wheels, thus making an equipment having a total generating capacity of 22 000 h.p.

The electric current from this station is stepped up to 23 000 volts and transmitted to Vancouver, New Westminster, Burnaby, Lulu Island, and elsewhere, and is used for operating electric railways,

salmon canneries, car shops, saw-mills, and for general municipal and industrial purposes, thus serving the requirements of an urban population of about 90 000.

The transmission line consists of double towers each carrying two circuits of 3-phase, No. 2, B. & S., copper cable, and crosses navigable

FIG. 1.

water at one place where there is a span of 2 750 ft. suspended 150 ft. above the water.

The total single 3-phase circuit length of lines is 135 miles, and the transforming capacity at present in service is 14 650 kw. at the generating station and 16 400 kw. at the various sub-stations; all

these transformers are air-cooled, and are of different sizes up to a maximum of 2 500 kw. per phase.

In the interior of British Columbia the sparse population—almost entirely south of the main line of the Canadian Pacific Railway—is engaged either in agriculture or mining, and thus it happens that, aside from small developments at Revelstoke and Nelson for municipal purposes, the only interesting hydro-electric plant in this district is that of the West Kootenay Power Company, built to serve the mines and smelters at Rossland, Trail, and in the Boundary District. It is located at Bonnington Falls, on the Kootenay River, 9 miles west of Nelson, B. C. For the new power-station—which supersedes a smaller adjoining station previously built—the design is of the vertical type, owing to the considerable variations in tail-race level, caused by gorging below the falls in high-water periods.

The hydraulic head varies from 65 to 55 ft., being least at high water, and the minimum capacity of the river is estimated at about 6 000 cu. ft. per sec., being the run-off from a mountainous drainage area of nearly 10 000 sq. miles, which, owing to the glacial sources of the river, gives a low-water period in winter and a high-water period in summer. The station is a very interesting example of complete concrete construction, including even the intake channels and draft-tubes of the turbines, and is constructed for an equipment of four units, each of 6 000 h.p., nominal rating, with the turbines of 30% overload capacity. At present, two units are installed and a third has been ordered, in addition to which the old station has an equipment of 2 700 kw., and, when required, serves Rossland and Trail over a 31-mile, cedar-pole line at 20 000 volts on two No. 0, B. & S., 3-phase circuits.

At the larger station, the triplex, high-pressure, oil-thrust pumps, etc., are developed very much along the lines of the Niagara works, and are designed to supply oil to the thrust-bearings at 250 lb. per sq. in.; the governors were built by the I. P. Morris Company, of Philadelphia, adapted and modified from the well-known Escher-Wyss type.

The step-up transforming station supplies 60 000-volt current for the new cedar-pole transmission circuits to Phœnix Mines and Grand Forks Smelters, and to Greenwood Mines, distant 79, 69, and 83 miles, respectively; and its present equipment consists of six single-phase,

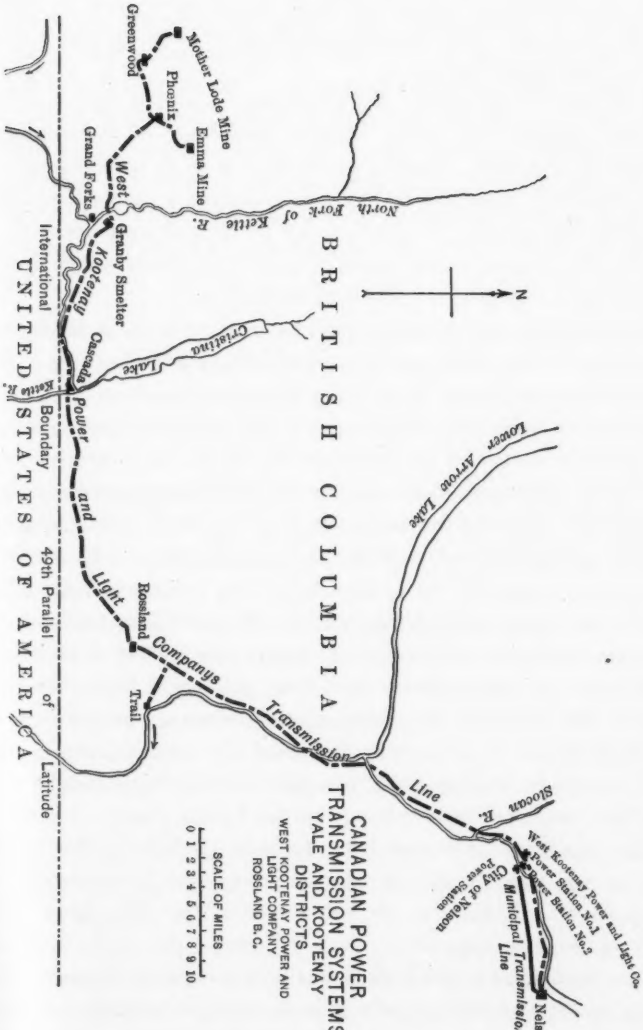
CANADIAN POWER
TRANSMISSION SYSTEMS
YALE AND KOOTENAY
DISTRICTS
WEST KOOTENAY POWER AND
LIGHT COMPANY
ROSSLAND B.C.
BRITISH COLUMBIA
UNITED STATES OF AMERICA
International Boundary
49th Parallel of Latitude
N
SCALE OF MILES
0 1 2 3 4 5 6 7 8 9 10
Kettle R.
Cascade Power and Light Companys Transmission Line
Trail
Rossland
Grand Forks
West Grand Forks
Greenwood
Phoenix
Mother Lode Mine
Emma Mine
Granby Smelter
Kootenay
North Fork of Kettle R.
Christina Lake
Lower Arrow Lake
Slocan R.
West Kootenay Power and Light Co. Power Station No.1
Power Station No.2
City of Nelson Power Station
Municipal Transmission Line
Nelson

FIG. 2.

water-cooled transformers, each of 1 850 kw., nominal rating, each three-phase bank being in a fire-proof compartment; there is also a bank of three single-phase, 1 250-kw. transformers held in reserve for the Rossland and Trail Districts at 20 000 volts in the event of a shut-down of the older station.

The uses for which the electric current from this station is adopted, in this district, are quite varied, consisting of mine-hauling by electric locomotives, hoisting by induction and synchronous motors, lead smelting, handling of various machinery, pumping, air compression, and general industrial and municipal purposes.

The total transforming capacity in seven sub-stations is 31 000 kw., and the active connected load is 21 000 h.p.

Immediately east of this hydro-electric development are located the great Crow's Nest Pass coal areas, but the competition is not effective, in so far as service in the mining district is concerned.

It is also anticipated that some of the adjacent mountain railways will become electrified in the near future, as the operation of rack-railways on 4½% grades has not proven satisfactory or economical.

Perhaps the most interesting feature of these works is the satisfactory maintenance of a 60 000-volt transmission line over a heavy mountainous country on a substantial but ordinary cedar-pole line. This is in sharp contradistinction to the prevailing tendency in the more level districts of Eastern Canada, where it is considered good practice to use steel towers with long spans for high-voltage transmission, but doubtless the decision was influenced by the inaccessible nature of much of the country traversed, the plentiful supply of large cedar timber in British Columbia, and also the high freight rates on structural steel, to the interior, from the Pacific Coast.

The adjoining zinc and lead smelters at Nelson, B. C., will be supplied by these works or by the municipal hydro-electric works of the City of Nelson, on the other side of the river from the large power-station just described.

This municipal power-station has now a capacity of 5 000 h.p. and an equipment of 1 000 h.p., which is about to be increased to 2 500 h.p. by the installation of a second vertical unit. The general construction has been carried out on a basis of four units having a total capacity of 5 000 h. p.

The Great Plains.

Immediately east of the Rocky Mountains, in Canada, there are two great river basins, with many subdivisions, but of these the Mackenzie River, emptying into the Arctic Sea, lies in a practically uninhabited region; however, it is interesting to note that emigration and settlement will soon change all this, and, undoubtedly, in a few generations populous communities will occupy the upper water-sheds of this river, and will require all the tools of civilization, and among them the electric current from hydro-electric stations. Flour mills are already being established on the head-waters of the Peace and Athabasca Rivers.

The Nelson River Basin.

The Nelson River drains an extensive water-shed which is a country in itself. All the inhabited portions of the Provinces of Manitoba, Saskatchewan, and Alberta, except a small area tributary to the Missouri and Mississippi Rivers, are drained by the various tributaries of this river.

This water-shed is divided into three main drainage areas, the Saskatchewan, the Red and Assiniboine, and the Winnipeg Rivers. From a hydraulic point of view, each of these rivers exhibits distinct and dissimilar characteristics.

The Saskatchewan River, of 125 000 sq. miles drainage area, rising in the eastern slope of the Rocky Mountains by many sources, fed from glaciers, and descending to the plains through many mountain gorges, offers, near its sources, reasonable opportunities for hydraulic development, and two companies are already active in the preliminary stages of contracting to supply Calgary, Alta., with electric power from hydro-electric stations to be erected on the head-waters of the Bow River. Doubtless, as population in the foot-hills increases, similar undertakings will be carried out to meet the requirements of near-by towns and cities.

Eastward of the foot-hills, however, the conditions change, and the main branches become chiefly dependent on the precipitation on the great plains, which is rather light and either evaporates or passes off in spring or autumn floods. The character of the country also changes; the streams occupy deep, wide gorges cut down into the plains, and have light grades and shifting silt and sand bottoms where dam construction would be difficult and expensive.

Speaking generally, the main valley of the Saskatchewan, eastward from the foot-hills, is not adapted to hydro-electric constructions, and, as there is coal over a large proportion of this vast area or near it, it is not probable that such constructions will ever be numerous, extensive, or profitable, except at a point on this river immediately before it enters Lake Winnipeg.

At this place, 250 miles north of the City of Winnipeg, the river passes through a series of small lakes and then, at Grand Rapids, drops quickly to the level of Lake Winnipeg. The change in level is 112 ft. in a few miles, and there is thus afforded the opportunity for an enormous development of at least 300 000 h.p. whenever the demand arises for a power development at that point.

The Red and Assiniboine branches of the Nelson River, with a drainage area of 105 000 sq. miles, are still more unfavorably located for hydraulic purposes, as the drainage basin is entirely in a plains country, where there is light rainfall and very little lake storage, thus the minimum run-off is extremely small in proportion to the areas drained, as a consequence of which, with the exception of a few quite insignificant opportunities, they may be written off the list of hydro-electric districts, not only for the foregoing reasons, but because the flood levels are usually bank full, and sand and silt bottoms, over wide flats, present expensive conditions for dam constructions. It may be remarked in passing that this drainage area is devoid of coal, except for the lignites of the Souris Valley; and the price of fuel and power is excessive, except where the coal supply is local or transmitted electricity from the eastward is possible.

The Winnipeg branch of the Nelson River drains an area of approximately 55 000 sq. miles in the State of Minnesota and in the Provinces of Ontario and Manitoba, and, from its peculiar topographical features, affords one of the most favorable locations for the development of hydraulic power in Canada, excelled perhaps only by the Niagara and the St. Lawrence and superior in some respects to the Ottawa River.

This condition is somewhat, but not greatly, offset by the moderate rainfall, which is only from 20 to 25 in. per year over the main area, but, aside from this, there is a most extensive network of lakes (of which the location, throughout the northern portion, has been, in many cases, unmapped until recently). There are thousands of lakes

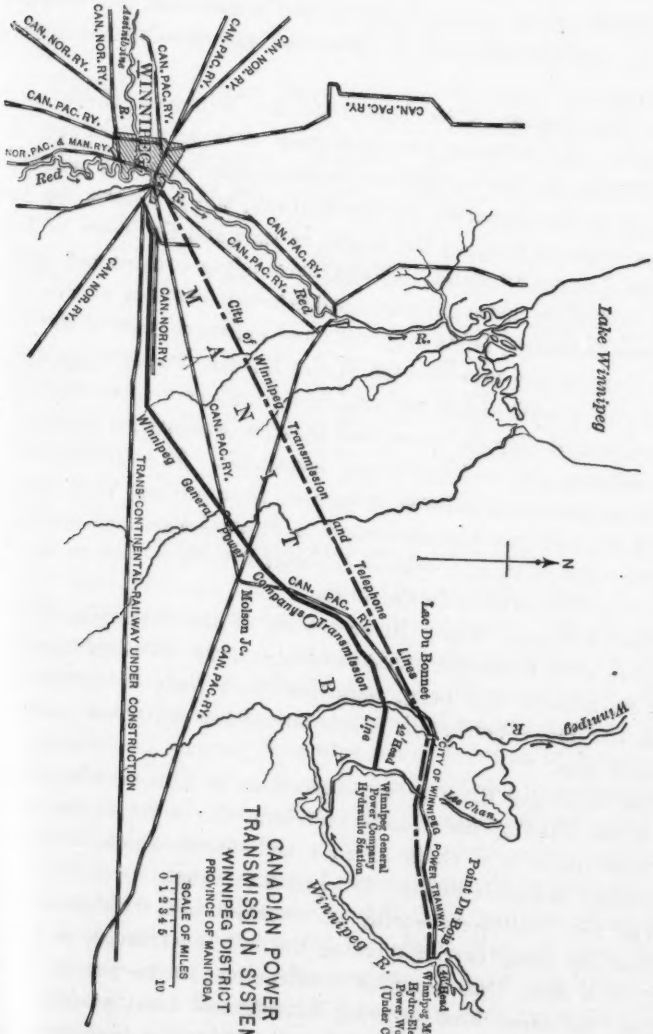
CANADIAN POWER
TRANSMISSION SYSTEMS
WINNIPEG DISTRICT
PROVINCE OF MANITOBA
SCALE OF MILES
0 1 2 3 4 5 10
WINNIPEG
CAN. NOR. RY.
CAN. PAC. RY.
CAN. PAC. RY.
CAN. NOR. RY.
CAN. PAC. RY.
CAN. PAC. RY.
CAN. PAC. RY.
CAN. NOR. RY.
CAN. NOR. RY.
CAN. PAC. RY.
NOR. PAC. & MAN. RY.
Red R.
Red R.
Red R.
Lake Winnipeg
CITY OF WINNIPEG
Transmission Line
Telephone Lines
TRANS-CONTINENTAL RAILWAY UNDER CONSTRUCTION
Winnipeg General Power Company
Molson Jc.
CAN. PAC. RY.
Lac Du Bonnet
Winnipeg R.
Lac Chaux
Winnipeg General Power Company Hydraulic Station
POWER TRAMWAY
Point Du Bois
Winnipeg R.
Winnipeg Municipal Hydro-Electric Power Works (Under Construction)
N

FIG. 3.

of all sizes, culminating in the Lake of the Woods. There are 1 600 sq. miles of water surface on the Rainy River branch and Lac Seul on the English River branch, westward of which the two rivers unite near the eastern boundary of Manitoba and flow thence into Lake Winnipeg.

On the English River branch of the Winnipeg River no large hydraulic constructions have yet been considered seriously except near Dryden, as the country is inhabited only at a few points along the line of the Canadian Pacific Railway, which crosses the various smaller branches between the height of land and the Town of Kenora, but the Transcontinental Railway, now being constructed, will pass much nearer the more extensive water-power locations of this river.

These locations, being only the more extensive ones of the English River branch, are rated by the Hydro-Electric Power Commission of Ontario at a total of 100 000 h.p., minimum dry-weather flow, 24-hour service, and it may be considered that the commercial capacity is at least 200 000 h.p. under ordinary conditions. It seems evident that the requirements of railway operation, flour milling, pulp and paper mill work, and possibly smelting will demand a careful study of the specific value of these powers in the near future, as soon as the Transcontinental Railway is in operation.

The Seine and Rainy River branch of the Winnipeg River also possesses good hydraulic opportunities, and the Koochiching Falls, at Fort Frances, is now being developed on a basis of 14 000 h.p., for which the dam construction is now completed and power construction is under way.

The Winnipeg River proper commences at Kenora, where the river leaves the Lake of the Woods, and from this point to the mouth of the river there is a chain of falls and chutes which, as sources of economical power, is difficult to surpass or equal.

The two conditions worthy of comment are that at each power location the rocky barrier creating the fall is admirable as a foundation for a dam, thus enabling constructions to be low in first cost, and at the same time creating considerable local storage to meet daily peak-load requirements. Also, the enormous lake storages, previously referred to, very much reduce the flood discharge and increase the minimum. The latter may be taken at 13 000 cu. ft. per sec. at Kenora and 20 000 cu. ft. per sec. below the junction of the English

River, the flood discharge being not more than four times these amounts.

Table 1 is a summary of the power sites on the Winnipeg River proper, between Kenora and the mouth at Fort Alexander.

TABLE 1.—Power Sites on the Winnipeg River.

Location.	Available head, in feet.	Minimum electric horse-power; 24-hour rating.	Average electric horse power; 24-hour rating.	Remarks.
Kenora	18	21 000	42 000	5 500 h.p. installed.
Island Falls	45	56 400	112 800	
Point du Bois	46*	83 600	209 000	20 000 h.p. being installed.
Slave Falls	30*	54 500	136 200	
Upper Seven Portages	30*	54 500	136 200	18 700 h.p. installed at Lee Channel.
Lower Seven Portages	22	40 000	100 000	
McArthurs Falls	12	22 300	55 750	
Grand Bonnet	30	56 000	140 000	
Little Bonnet	10	18 500	46 200	
White Mud Falls	15	28 000	70 000	
Silver Falls	21	39 100	97 800	
Pine Portage Falls	12½	23 300	58 300	
Totals		497 200	1 204 250	

* Including raising the water level by head-works.

Of the sites mentioned in Table 1, Island Falls, Point du Bois, Slave Falls, Upper Seven Portages, and Grand Bonnet afford excellent construction opportunities. All but the first two in Table 1 are within easy transmission distance of the City of Winnipeg, and all are within short distances of two transcontinental railway systems.

The situation at Kenora illustrates the great value of water-power as applied to flour milling. The town has little else to recommend it as an industrial center, not being at a competing railway point, nor at a bulk-breaking point; neither is there any surrounding population or any fertile areas; but, in spite of these apparently unfavorable conditions, three large flour mills are located there, and use about 4 000 h.p., with a capacity of 10 000 bbl. per day. Two of these plants operate privately-owned hydraulic equipments, and the third purchases 1 000 e.h.p., at a flat rate of $10 per e.h.p.-year, from the municipal power-station. The latter has been built recently at the eastern outlet of the Lake of the Woods with dimensions to receive machinery having a capacity of 4 000 e.h.p., one-half of which has already been installed.

At the main or western outlet of the Lake of the Woods, at Kenora, there has also been built for some years, a stone dam from which water could be drawn to develop approximately 18 000 e.h.p. at low-water, but no further constructions have been commenced.

At Point du Bois the City of Winnipeg (population 120 000) has commenced the construction of a large generating station involving a 75-mile transmission line to the city, and the designs contain some interesting features. A short (1 200-ft.) head-canal, concrete overflow walls, and a rock-filled dam in the main river will raise the water 13 ft., creating a total head of 46 ft. and forming a lake 5 sq. miles in area, thus producing uniform and quiet hydraulic conditions. The dimensions of all control works, dams, etc., provide for an ultimate equipment of 60 000 h.p. The remainder of the work is to be a solid reinforced concrete structure involving an enclosed forebay room, power-house, and transformer-station equipped primarily with five 5 300-e.h.p. units, which will consist of horizontal, double-runner turbines (draft-chests only) in reinforced concrete wheel-pits and directly connected through a breast wall to 3-phase, 60-cycle, 6 600-volt, generators making 164 rev. per min.

The transformation will be to 77 000 volts, by water-cooled, oil-insulated transformers having a capacity of 3 750 kw. per phase. Similar transformation at Winnipeg will be from 72 000 volts down to 11 000 volts in a main terminal station, from which 11 000-volt feeders will distribute to three or more sub-stations for local distribution at suitable voltages; the transformers at the terminal station, however, are to be oil-circulating, and oil-cooled. The system of remote control will be fully applied at both stations.

The transmission lines, of No. 000 copper, or aluminum equivalent, will be carried on double-circuit steel towers spaced 600 ft. apart, each alternate tower being of elastic type with two legs only. All tower foundations are to be on concrete.

The city utilities are already numerous, and include Artesian water supply, pumped electrically; and the desire to be able to offer cheap power to industries, present and prospective, which would otherwise labor under a great disadvantage in a city which cannot offer bituminous coal of a good quality for less than $6.50 per ton, while anthracite coal is $10.50 per ton retail, appears to be ample justification for this ambitious project for a city of 120 000 inhabitants.

The Winnipeg Street Railway Company and its allied companies have in operation a hydro-electric station on the Lee Channel of the Winnipeg River, and a transmission line 67 miles long supplying power to a terminal station in the city.

This generating station was selected apparently without proper investigation and without an eye to the future, and the company is already expending large additional sums on the tail-race channel in order to insure sufficient hydraulic head and flow of water at low water to operate the machinery now installed, which makes a completed scheme incapable of further expansion and consisting of nine units having a combined rated capacity of 14 000 kw.

The scheme of development consists of a rock-filled dam across the Winnipeg River, which raises the normal water level 4 or 5 ft. and forces a flow of water through a 14-mile high-water channel, from which already approximately 500 000 cu. yd. of rock have been excavated.

The power-house is about 5 miles from the intake of this channel, and was designed for an operating head of 42 ft.; this, however, has not yet been realized, and it is understood that, when delivering 15 000 e.h.p. at Winnipeg, the generating station is only operating under a head of 36 ft., and is at its present limit of capacity, hence the further excavations now in progress.

The generating station has an open forebay, steel-flume wheel-pits, and units of five 2 000 kw., and four 1 000 kw., each operated by two twin horizontal turbines; the generators are for 3-phase, 60-cycle, 2 300-volt current.

The transformation is to 50 000 volts, and the transformers are oil-insulated, water-cooled, of 1 800- and 830-kw. capacity per phase, each being in a separate compartment. The total transformer capacity at the generating station is 19 500 kw. The transmission system is on double-circuit, steel towers, spaced 500 ft. apart, with protruding pieces of angle iron on each upper corner of each tower which act as lightning arresters. It is understood that this protection is very satisfactory, there having been only one shut-down during the first year's service (which commenced in 1906). The capacity of the terminal station in Winnipeg is equal to that of the transforming station at the generating works.

The utilities served consist of the city and suburban street rail-

ways, the city house-lighting, and current for the present municipally-operated street lighting. In addition to this, there is a small distribution of power for miscellaneous purposes, and two large blocks of power for flour milling. The capacity of the generating station is already fully taxed during the low-water winter season, and no effort is made to encourage large users of power by offering attractive rates; this power-station cannot be looked upon as a serious competitor to the municipal power-station now being built.

The three tributaries of the Nelson River, which have just been discussed hydraulically, converge in Lake Winnipeg and there form an enormous reservoir of 9 300 sq. miles at an elevation of 710 ft. above the Hudson Sea.

When it is considered that the drainage area feeding the outlet of Lake Winnipeg into the Nelson River proper is nearly 350 000 sq. miles, and that the water supply comes from three different and dissimilar climatic and topographical river basins, one can appreciate what a large minimum flow there is; and, as the route of the river to the Hudson Sea is rock-ribbed, in cascades, pools, and waterfalls, all that need be said here is that, should electro-chemical or other demands require very extensive water-powers, where relation to habitation is immaterial, these, to the extent of at least 4 500 000 h.p., can be readily secured on the Nelson River between Lake Winnipeg and the Hudson Sea.

Other Hudson Sea Rivers.

Of the many extensive rivers flowing into the Hudson Sea and James Bay, there is very little accurate information, and hydraulically these rivers are almost unknown except in a general way.

It can be appreciated, however, that, from the physical conformation, many enormous water-powers exist, because the table-land sources of the rivers abound in lakes at an elevation of from 600 to 1 500 ft. above the sea, and this, taken with a good rainfall and the rocky nature of the country, warrants this certain assumption. It is generally known, however, that near the sea coast most of these rivers are alluvial, and that the water-powers of value lie considerably inland, in general, at least 200 miles from the mouths.

The first development, north of the height of land which lies between the St. Lawrence River and the Hudson Sea, will probably be at Iroquois Falls, on the Abitibi River. The construction of the

Ontario Government Railway, parallel to the river, and that of the Dominion Government Transcontinental Railway east and west immediately north of Lake Abitibi, suggest the early development of Iroquois Falls for purposes of pulp grinding, or power for mines, or for general distribution to the junction city which will immediately grow up in that vicinity.

The St. Lawrence Basin.

The St. Lawrence River drainage basin is the chief seat of population in Canada, and naturally the hydro-electric developments of this region are most extensive.

The total drainage area is 565 000 sq. miles, but of this about 90 000 sq. miles comprise the water surface of the Great Lakes, which form enormous compensating reservoirs, their regulation and control, international in character, presenting a very delicate physical and political question, additionally complicated by the rights and needs of navigation; therefore it is advisable to discuss the water-powers of the main river and of its many important branches separately.

Sault Ste. Marie.—The first water-power location on the main river is at Sault Ste. Marie where Lake Superior, with 31 000 sq. miles of water surface, empties into Lake Huron, falling approximately 18 ft. and discharging about 60 000 cu. ft. per sec. at low water, thereby affording opportunity for about 90 000 e.h.p., after making allowances for various hydraulic losses. The present developments have been somewhat international in character, and have been carried out chiefly by the Lake Superior Power Company, which, with allied companies, disposes of 6 500 h.p. of hydraulic and electric power on the Canadian side of the river (out of a designed total of 20 000 h.p.) to rail mills, pulp mills, and for industrial and municipal purposes in the Canadian city.

On the United States side of the river the same company constructed a canal and power-house, under unfavorable physical conditions, designed to obtain 45 000 h.p.; but machinery for only one-half of this, or less, has ever been installed, and the chief customer, outside of municipal and small industrial users, is a carbide company having works in the vicinity; a third company, on the United States side, has a small hydro-electric development, but it is probable that no further concessions will be granted there, or, if granted,

will be under the control of the International Deep Waterways Commission, so that the needs of navigation, both as regards the maintenance of the level of Lake Superior and the water required in increased quantities for operation of the three locks at this point, may be fully conserved.

It is worthy of more than passing comment that, although these canals and locks serve an enormous commerce, their existence has been of little local value, whereas the life of the two adjacent cities of Sault Ste. Marie depends chiefly on the creation and operation of hydro-electric power, and the two Governments interested will be well advised to use every effort to facilitate rather than hinder the power use of all the water passing from Lake Superior, which is not required for locking purposes. It is evident that control works at the head of the rapids can be constructed so as to compensate for any quantity of water which may be diverted and used for power purposes, and, geographically, as an assembly point for raw materials by water and rail, Sault Ste. Marie has many advantages.

Niagara Falls.—Continuing seaward from Sault Ste. Marie, the next location which affords hydraulic opportunity is spoken of familiarly as Niagara Falls.

Lake Erie flows into Lake Ontario by a river route which, from Chippewa to Queenston, is a series of rapids and the cataract, but the cataract of Niagara Falls is barely one-half of the total difference in elevation, which is 327 ft., and that the first great hydro-electric developments have been made there with effective heads of from 136 to 175 ft. can only be looked upon, in the future, as an economic blunder, because there are potential developments which can be made at several locations in the United States and Canada using a net head as great as 300 ft., which, for as large an installation as 200 000 h.p. or greater, would not be as expensive as those now constructed, and the deduction is here offered that when industry makes the demand neither international understandings nor the makeshifts of opportunist politicians will prevent capital from carrying out the construction of these undertakings for the benefit of mankind.

This question of utilizing the hydraulic power of the Niagara escarpment is, in reality, a world problem, not a local one, because there is there latent 5 000 000 continuous electrical horse-power, considering only the low-water flow of 180 000 cu. ft. per sec. at an

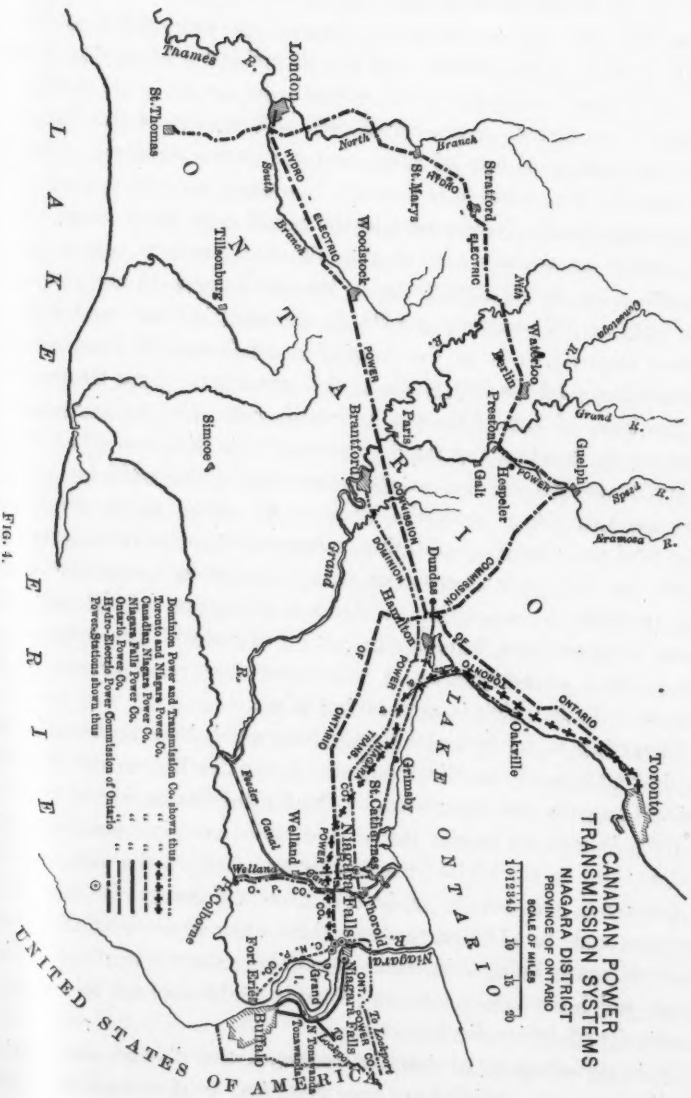
CANADIAN POWER
TRANSMISSION SYSTEMS
NIAGARA DISTRICT
PROVINCE OF ONTARIO
SCALE OF MILES
10 0 10 20
LAKE ONTARIO
LAKE ERIE
ONTARIO
UNITED STATES OF AMERICA
Thames R.
London
St. Thomas
Tillsonburg
Simcoe
Woodstock
St. Marys
Stratford
Paris
Brantford
Galt
Preston
Hespeler
Waterloo
Guelph
Dundas
Hamilton
Oakville
Toronto
Grimsby
St. Catharines
Niagara Falls
Welland
Port Colborne
Fort Erie
Buffalo
Grand R.
Speed R.
Bramosa R.
Dominion Power and Transmission Co. shown thus
Provincial Power Co.
Canadian Niagara Power Co.
Niagara Falls Power Co.
Ontario Power Co.
Hydro-Electric Power Commission of Ontario
Power Stations shown thus

FIG. 4.

effective head of 300 ft., and, estimating even so little as 2½ lb. of coal per horse-power-hour, this would be equivalent to a saving of 55 000 000 tons of coal per year, should this great force of Nature ever become so fully utilized; and why should it not be used in increasing amount as required? There will still be available for scenic purposes all the surplus water, which will still pass over the falls, even after the minimum has all been used for power purposes. Consider the question otherwise; this location is unique, as it is on one of the greatest highways of commerce in the world, both water-borne and by rail, and it is in the midst of a fertile and populous region, which, for 150 miles in all directions, is becoming more thickly populated year by year. The steady growth in the electric load carried by the present stations leads to the inevitable conclusion that we are only in the infancy of the utilization of this great gift of the Creator.

The level of Lake Erie can be readily controlled by compensating works at the head of the rapids, and might be designed so as to constrict the natural channel at the same rate as the artificial channels were created, either near the river or by other routes from Lake Erie to Lake Ontario, and this arrangement appears to meet and answer the only serious criticism which has tended to array the shipping interests in opposition to further utilization of the water for power purposes; and, indeed, diminution of natural flow presents certain positive advantages, as it will check that rapid erosion of the Horseshoe Falls which is now taking place.

Recurring to the actual constructions which have been made since the first station of the Niagara Falls Hydraulic Power and Manufacturing Company was completed, on the United States side of the river, in 1881, it may be briefly stated that there are six large generating stations, two on the United States side and four on the Canadian side of the river, engaged in producing electrical energy for commercial purposes, and, in addition to these, there are numerous small stations, one at Niagara Falls, Ont., and several along the route of the Welland Canal, which produce a total of probably 15 000 h.p. for miscellaneous electrical and hydraulic purposes.

It is unnecessary to describe in detail the diverse constructions, distributions, etc., which have been described so often and so minutely by various engineers, but certain broad facts will be given by which the scope of present accomplishment may be gauged.

On the United States side, the Niagara Falls Hydraulic Power and Manufacturing Company has the distinction of being first in the field, and, from diminutive beginnings in 1872, it has expanded its works until at the present time it has 77 500 e.h.p. installed, and building construction is under way for additional equipment having a capacity of 70 000 e.h.p. In addition to this, water is supplied to various industries developing 8 000 h.p. of direct hydraulic power for pulp, paper, flour, and other mills.

The power from these works is used for street railway and various industrial and electro-chemical purposes, but chiefly for the manufacture of aluminum, by the Pittsburg Reduction Company, which company has here its largest works. It is understood that this company contracts for nearly 60 000 h.p.

The head canal for this development is 110 ft. wide and 14 ft. deep at normal water level. It is constructed through the City of Niagara Falls, N. Y., for a distance of 4 400 ft., and terminates at a forebay immediately overlooking the river. The water is used under a total head of 210 ft. and a net head of about 200 ft., and is led down the side of the cliff in steel penstocks to the power-stations at the lower river level, where is installed a heterogeneous collection of direct-current and alternating-current generating units composed of various sizes from 150 to 1 000 kw. in the older station and from 3 500 to 6 500 kw. in the newer station, which facts well illustrate the growth of the art and the magnitude of the business of this station, which is entirely local, involving no transmission lines of any importance.

The Niagara Falls Power Company has two generating stations operating under an average effective head of 140 ft., one on each side of a short head-canal, and the common tail-race is a brick-lined tunnel passing to the lower river, 7 000 ft. distant.

These stations, heralded at the time as pioneers, have long since been eclipsed by the stations on the Canadian side of the river, and also suffer from certain engineering defects which, no doubt, were unforeseen at the time of construction and have since been cumulative.

The ice difficulties, after years of strenuous effort, have been largely overcome, by improvements in the intake works and by the addition of an ice shoot to the tail-race tunnel, but the first station, having a capacity of 50 000 h.p., has unprotected ice racks and is equipped with free, outward-discharge, double-runner, vertical tur-

bines. The ten generators are each 5 000-h.p., 2-phase, 2 200-volt, with revolving armatures. The second station, having a capacity of 60 000 h.p., has additional submerged-arch entrances and an enclosed forebay for further protection from ice. The turbines are of the single-runner, vertical type, with draft-tubes; the eleven generators are each 5 500-h.p., 3-phase, 2 200-volt, and are partly of revolving-field and partly of revolving-armature types. The tail-race tunnels are led from the power-stations, by very sharp curves, to a junction, and this, together with the back-piling in Power-house No. 1 due to the use of free-discharge wheels, makes it impracticable to run both stations to full capacity. It is understood that 83 000 h.p. is about all that is generated, although the market in Buffalo is being largely abandoned to the Canadian Niagara Power Company, its station being several miles nearer and belonging virtually to the same financial interests.

The power from these stations is now distributed in large blocks, chiefly to local electro-chemical consumers, and the policy of the company in creating a manufacturing district, with railway switching facilities, building sites, etc., and giving long-term power contracts, with guaranties of continuity, under penalty, has created a large and stable industrial center consuming more than 60 000 h.p. continuously from those two stations. The power transmitted to Buffalo, Tonawanda, etc., is 22 000-volt, 3-phase, 25-cycle. The pole lines are of ordinary wooden type and carry four 3-phase circuits, two on each line, some circuits being of copper and some of aluminum. Until the load was transferred to the Canadian station, all the varied electrical utilities of Buffalo, to the extent of about 40 000 h.p., were operated from these United States stations, as are still the various interurban and scenic electric railways converging at Niagara Falls.

On the Canadian side of the Niagara River four large generating stations have been constructed recently; three of these are at Niagara Falls, and the fourth is at a point 12 miles westward.

The Dominion Power and Transmission Company (formerly the Hamilton Cataract Power Company) first commenced to deliver power to Hamilton, Ont., in 1897, from a small station 12 miles westward of Niagara Falls at the foot of the Niagara escarpment, and drawing its water supply from the Welland Canal and thus from Lake Erie.

This hydro-electric development utilizes a hydraulic head of 267 ft.,

and is a very economical one, as excessive expenditures for head-works were not required. This enterprise has a progressive management, and a growing market for power has expanded it until, at the present time, the machinery equipment, including transformers, has a nominal rating of 38 400 h.p.

It is an interesting feature of this generating station that the turbines are entirely of European manufacture and design, being supplied by Italian and German makers, as the earlier experiences with manufacturers in the United States were not satisfactory.

The electric current is generated at 2 400 volts, 66⅔ cycles, and is transmitted to the surrounding towns and cities at various voltages. The chief market is in Hamilton, 32 miles westward to which three pole-line, 3-phase circuits, two of copper and one of aluminum, carry power at 40 000 volts. In addition to this, however, the company serves a network of street and interurban railways which it owns or controls, and supplies power and light in Brantford, Welland, St. Catharines, Dundas, etc., with a total of 132 miles of transmission line.

This company, owing to its favorable investment and strongly entrenched position, is able to distribute with profit at low rates for power, and has thus built up a large business very rapidly. It is understood that the peak load in Hamilton alone, which is a city of only 60 000, is considerably more than 20 000 e.h.p., which includes current for operating street and interurban railways owned by this company, and also cotton mills, knitting mills, flour mills, rolling mills, and machine shops; in fact, every phase of a busy manufacturing city is provided for.

This company also owns small hydro-electric generating stations at St. Catharines and Brantford, and may be said to control, almost exclusively, the business of supplying electricity in the Niagara District west of Niagara Falls and Thorold.

The generating station of the Canadian Niagara Power Company was constructed between 1901 and 1905. This company is practically identical with that of the Niagara Falls Power Company, and although it distributes some 2 000 to 3 000 h.p. locally to various industries in Niagara Falls, Ont., it has not entered into the long-distance transmission field, no doubt considering the local sale to be ultimately more profitable. The generating station is of the wheel-pit type, with a tail-race tunnel 2 200 ft. long, and operates under an

effective head of only 140 ft. The water is taken from the rapids immediately above the cataract, and although protecting works, submerged-arch entrances, enclosed forebay room, and sluiceways have been constructed to meet the difficult ice conditions, they are only moderately satisfactory, and a continuous winter service is maintained only by strenuous efforts and with considerable difficulty.

The generating works are complete for eleven units, each having a nominal rating of 10 000 e.h.p., and five of these machinery units are now installed.

The turbines, of Swiss design, are of the vertical type, with draft-tubes and double runners, and the generators are 12 000-volt, 3-phase, 25-cycle.

This station is inter-connected with those of the Niagara Falls Power Company by a large number of 11 000-volt underground cables, thus warranting great certainty of service; but the chief market for power is in Buffalo.

The company has a transformer station, adjoining the generating station, with an equipment of fifteen single-phase, 1 250-kw. transformers, all mounted in one room; and although these are arranged so as to be operated, if desired, at various voltages up to 60 000, the pressure used at present is 22 000.

The transmission system consists of steel poles, carrying two 3-phase circuits of aluminum cable mounted on cellulose insulators, and a third circuit is now being built. The capacity is 12 500 h.p. per circuit at 22 000 volts. At Fort Erie this transmission system has two spans of 1 600 and 2 200 ft., respectively, suspended 130 ft. above the water of the Niagara River, and it is understood that considerable difficulty was experienced in cable breakage until elastic counter-weighted suspensions were made at the end towers with suspended insulation at the center tower.

Under the franchise conditions existing between this company and the Provincial Government, and having in view the water limitations now being reported on and recommended by the International Deep Waterways Commission, it would appear to be inevitable that this company will soon be forced to enter actively into exploitation of the Canadian market, in order to enable it to increase its output sales beyond the present, which not only amounts to about 40 000 h.p. from its own station, but to this must be added some 5 000 h.p. which is

being obtained from the Electrical Development Company and exported.

It is interesting to note that the combined output of the three stations of these allied companies, *i. e.*, The Niagara Falls Power Company and the Canadian Niagara Power Company, is more than 500 000 000 kw.hr. per year, and that the load factor is more than 70%, owing to the large number of electro-chemical customers.

Between 1902 and 1906, the Ontario Power Company constructed, immediately below the cataract, a station which is equipped at present with six horizontal units, each of 10 000 or 12 000 e.h.p., generating 3-phase, 25-cycle, 12 000-volt current, at 187½ rev. per min., under a net head of 178 ft. The turbines are of German design and manufacture. The water is brought to the station by an 18-ft. steel pipe, 6 500 ft. long, which is embedded in concrete just below the hydraulic grade line, and buried in Queen Victoria Park a few feet beneath the surface.

This company selected a favorable location for its entrance works and forebay, which are located at the head of the rapids and are completed for a capacity of 180 000 h.p., and, as far as ice troubles are concerned, it may confidently expect to give a continuous service.

In several other features these works are interesting, particularly in respect to the methods of control. To handle successfully an 18-ft. column of water, 6 500 ft. long, moving at 15 ft. per sec., under ordinary commercial electrical conditions, was no mean task, but it is understood that, by means of a spiral overflow structure, this has been accomplished. The electrical control is centered in the transforming and distributing station, which is 600 ft. from the generating station and on the high bluff overlooking the cataract. At this point all the current from the generators to the high-tension lines is controlled. The transforming station is equipped with eighteen single-phase, oil-insulated, water-cooled transformers, each of 3 000 kva., each bank being in a separate compartment. This company has created several allied companies which distribute 62 500-volt and 12 000-volt current in the United States and Canada. Its extreme eastern delivery is at Syracuse, N. Y., 161 miles distant, and it distributes current westward to St. Catharines, Welland, and Port Colborne; but in view of the contract it has now entered into with the Hydro-Electric Power Commission of Ontario for the supply of from 25 000 to 100 000 h.p. at from $9.00 to $9.40 per h.p. per year for 12 000-volt continuous power,

it appears evident that its sphere of influence in Southwestern Ontario is in the near future to become very considerable, and the output of its station increased to at least 50 000 h.p.

At present its maximum load is 25 000 h.p. in the United States and 5 000 h.p. in Canada, sales being at the rate of 110 000 000 kw-hr. per year.

This company's transmission lines are very extensive and varied in character, and comprise several hundred miles of double-circuit and single-circuit, aluminum lines, chiefly mounted on steel towers carrying one circuit each. Some of the branch lines and 12 000-volt lines, however, are on timber, **A**-frame towers or cedar poles; the various constructions have been carried out substantially and scientifically, but are subject to the criticism of transmitting at only 60 000 volts for distances up to 160 miles, whereas, at the present time, 80 000-volt transmissions are in use and 100 000-volt projects are under construction; however, it is true that the suspended insulator has not yet been fully developed to unquestioned satisfaction, and, at 100 000 volts, air emanations from conductors, line troubles, etc., are certain to be considerable.

In 1903 the Toronto and Niagara Power Company was formed by parties interested in the Toronto Street Railway Company and the Toronto Electric Light Company, and, after negotiations to buy electric current from other Niagara companies had failed, a lease was secured from the Provincial Government to build a generating station in Queen Victoria Park at Niagara Falls at a site lying between those of the Canadian Niagara Power Company and the Ontario Power Company, the works of both these companies being then under construction. The location chosen was notable in two particulars, the intake being almost wholly in the surging rapids above the cataract, while the tail-race tunnel was located so as to emerge beneath the cataract itself. The construction of these works proceeded from 1903 to 1907.

Preliminary to construction, it was most difficult to construct the coffer-dams and keep them water-tight, and, together with the extensive overflow dam, etc., afterward constructed, made a heavy investment necessary. The wheel-pit is constructed for eleven units, each of 8 000 kw., and four of these units are now installed. The operating head is about 140 ft. and the station is equipped with

Francis double-runner turbines, constructed by United States manufacturers on Swiss models. In various items, however, these turbines differ from previous constructions. The draft-tubes are carried outward to twin, parallel, tail-race tunnels, and emerge from underneath so as to maintain the water seal. The penstocks are alternately right and left, in groups of two, thus economizing in wheel-pit length. The supporting structures for the vertical main shafts of the units are concrete arches instead of steel girders. Altogether, these works give evidence of advance in the art of wheel-pit construction. The power-house is a most ornate structure; indeed, this company has carried on all its operations with a lavish hand, and a resulting large investment. Its generating company is known as the Electrical Development Company, and the 3-phase, 25-cycle, 12 000-volt current generated is delivered outside of Queen Victoria Park to a transformer station whence the Toronto and Niagara Power Company transmits it at 60 000 volts to Toronto, 80 miles distant (population 300 000), over a double-circuit steel-tower line. This line is carried across Hamilton Beach and Canal and the Welland Canal by pairs of very high and heavy towers; otherwise the transmission line passes through a level cultivated country without serious transmission difficulties, although including various spans across small valleys of more than 1 000 ft. each.

The transforming stations are built apparently with a great effort to attain safety, each single-phase transformer being in a separate compartment, and there is very extensive concrete cell construction for bus-bars, lightning-arrester compartments, etc. The Toronto terminal station also has an auxiliary water-tower for supplying water to transformers in addition to the water service from the city mains. The current delivered at this station at 55 000 volts is stepped down to 11 000 volts and is then carried by underground feeders to various sub-stations in the city where it is transformed into 600-volt, D. C. and various voltages, A. C.; also, the frequency is changed from 25 to 60 cycles. Altogether, from 20 000 to 22 000 e.h.p. are used by the two subsidiary companies, in addition to sales at Niagara Falls to the Canadian Niagara Power Company for export, as before mentioned, and there are also subsidiary companies formed to distribute eastward in the United States.

This brief description of the capacities of the various Niagara

generating stations, amounting to 450 000 e.h.p. in Canada, when the stations described are fully equipped, and the fact that several other companies own charters and water rights, and are anxious to enter the field if given opportunity and encouragement, at first, would suggest that electric energy can now be obtained in this favored district at very low rates, and, comparing the rates offered with the cost to any given consumer to produce power or light by steam-driven prime movers, certainly the rates do improve these conditions somewhat, as is shown by the steady growth of electrically-operated industries in Hamilton, Ont. Moreover, from $20 000 000 to $25 000 000 have already been invested by these four companies in generating stations and transmission systems which must find a return by the sale of the electrical output, but, for the last ten years, the people of Southwestern Ontario have steadily asserted that, to create a great industrial area in that part of the province, still cheaper power ought to be and could be obtained.

Boards of trade began to agitate in 1900, and in 1902 a committee, from various cities and towns, was formed to take concrete action, which, by pressure on the Provincial Government, induced the passage of an Act in 1903, enabling two or more municipalities to appoint a Commission and, after investigation, to carry out, if desired, the construction of the necessary works at Niagara or elsewhere for generating, transmitting, and distributing electric energy.

This municipal commission was appointed, made a very exhaustive investigation, and reported in 1906 to the seven municipalities then represented in the movement, but, owing to the limitations evident in the working of the Act, and to a larger movement having commenced, there has been no specific outcome from this report.

In 1905 the Provincial Government, becoming more fully alive to the importance of this question, appointed a Government Commission, which was instructed to study and report on the hydraulic possibilities in the province, the industrial demand then existing for electric energy, and the prime cost of generating and delivering the same to the various industrial centers and customers. The reports arising from the investigations were presented, in due course, to the Legislature during the session of 1905-06, and, although the facts elicited furnished a comprehensive idea of the industrial condition then existing, and the cost of satisfying the same, and also, by comparison,

set forth the rates offered by distributing companies, it has only been after a subsequent protracted struggle that definite results have been obtained and the evident will of the people carried out.

The vested interests immediately attacked the reports and the *bona fides* of those who drew them up, but the Provincial Government, recognizing the demand made upon it, passed a very comprehensive Act, appointing a permanent Government Commission. In this commission was vested the rights of the Crown, and it was authorized to act in response to the demands of the municipalities and in their behalf.

This Commission, after 2½ years' existence, has justified itself; and a group of sixteen municipalities has made a demand upon the Commission to secure electric power, to construct transmission lines from Niagara, to transform the electric energy thus transmitted at the various sub-stations, to administer the construction, and to operate the works for a period of 30 years, over which period a sinking fund will be created out of the rates to repay the investments. The amount of power to be distributed immediately is approximately 30 000 e.h.p., but it is confidently expected that this will increase very rapidly to probably 100 000 e.h.p. in 5 years.

The various municipalities have voted the money necessary for local distribution, and the Government has recently contracted for the construction of 300 miles of transmission lines, and this work is to be followed immediately by the construction of the necessary sub-stations and local distributions.

This popular movement has yet to be justified by capable and honest administration, deficiency in which is the weakness of many municipal movements in America, and it is to be hoped that the ægis of the Government Commission will render the inherent merits of the project effective.

The large area of country to be covered and the necessary anticipation of a still wider area as soon as the first installation has justified itself, have forced on the Government engineering staff, a consideration of higher voltage, and, according to present plans, the ultimate voltage of transmission now being provided for is 100 000. This advanced practice, however, has been adopted by reason of the necessary provision for further extensions of the Hydro-Electric Commission's system to points still more distant from the generating station

than London and St. Thomas (140 miles). The world will, doubtless, watch closely the results obtained in carrying out this huge municipal-provincial distribution of electric energy.

St. Lawrence River.—The St. Lawrence River, between Lake Ontario and Montreal, may be considered as having its navigation interests paramount, and, although attempts have recently been made to obtain permission from the Canadian Government to devote the whole river to power development, it is not likely that such permission will ever be granted.

In the meantime, along the ship-canal system, many small installations have been permitted, but they are chiefly local in character and are devoted to operating mills and factories. They amount roughly to 10 000 h.p., and, in addition to this, hydro-electric stations having a capacity of about 5 000 h.p. supply the various near-by towns with light and power, and light the ship canals during the navigation season.

The whole river, if available, would be a great source of power, as it is capable of developing, at low water, 1 500 000 h.p. of electric energy at the various rapids.

Montreal.—In the vicinity of Montreal (population 400 000), several interesting constructions have been made at various times by various companies, but finally all these interests have been merged into the Montreal Light, Heat and Power Company which controls the situation at present, although a rival company has recently been organized to develop power on the abandoned Beauharnois Canal of the St. Lawrence system and transmit to Montreal and distribute in opposition to the Montreal Light, Heat and Power Company. The latest construction by the latter company is at Soulanges, on the Soulanges Canal of the St. Lawrence River, 27½ miles west of Montreal, where 40 000 cu. ft. per sec. are available under a head of 54 ft. The equipment now being installed consists of three units, each of 3 750 kw., which will generate 3-phase, 60-cycle current at 4 000 volts. The transmission will be on one circuit at 44 000 to 40 000 volts, and the transformation will be made by banks of single-phase, water-cooled transformers having a rated capacity of 2 500 kw. per phase.

At present, this company owns two water-power stations and four steam stations, the latter having a capacity of 9 000 h.p. The com-

pany also purchases 16 000 h. p. delivered in Montreal by the Shawenegan Water and Power Company.

The station at Lachine, on the St. Lawrence River, is an example of daring but mistaken construction, carried out 12 years ago, when the transmission problem had not been as fully worked out as at

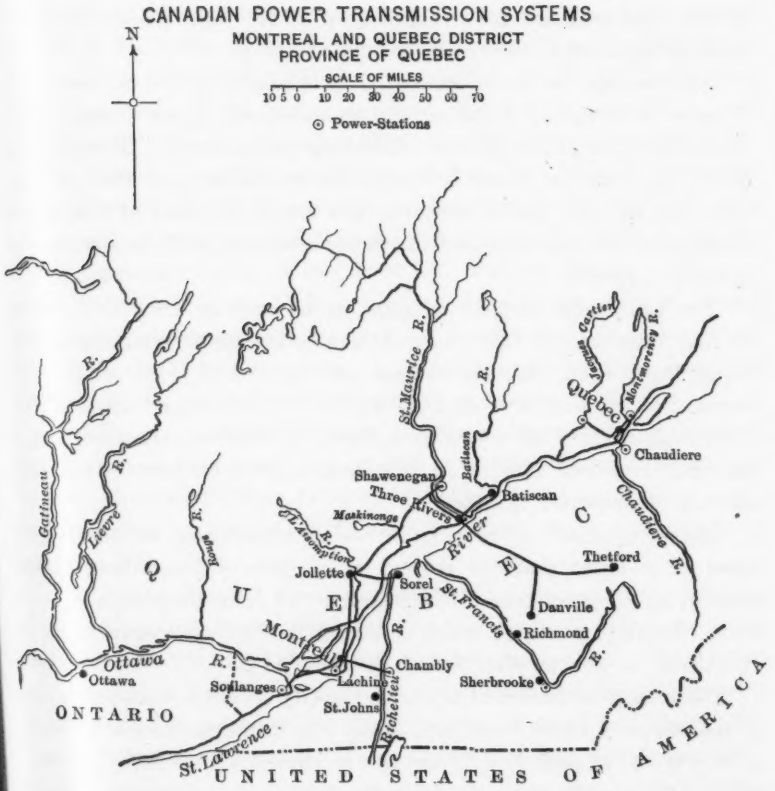

FIG. 5.

present. The works are really constructed as a part of the river, and therefore, in operation, they are subject to troubles from frazil in the head- and tail-races; but, after excessive expenditures, made for the purpose of creating quiet conditions above the power-house and protecting the tail-race from back-water, the station can be reasonably depended on to give continuous winter service.

The dependable hydraulic head is only 14 ft., and the eight generators, each having a rated capacity of 750 kw., are operated by groups of six turbines each, driving jack shafts.

The current generated is 3-phase, 60-cycle, at 4 400 volts, and is transmitted over eleven circuits at this voltage direct to Montreal (6½ miles), where it is stepped down to 2 400 volts for distribution. The station was designed for a capacity of 16 000 h.p., but it has never been equipped with machinery beyond 8 000 h.p.

At Chambly, on the Richelieu River, a branch of the St. Lawrence, 18 miles southeast of Montreal, the company owns a generating station which has also had a history interesting to engineers. The concrete dam, constructed to create a head of 34 ft., failed soon after completion, and the first year's operation was much disturbed by frazil, but a new dam has since replaced the faulty structure, and the station now operates normally.

The equipment consists of eight units, each of 1 800 kw., driven by four turbines per unit, the total generating capacity of the station being 20 000 e.h.p.; but the average load carried is 15 000 e.h.p. The current, which is 2 200-volt, 2-phase, 66-cycle, is stepped up to 25 000 volts, 3-phase, by 2 750 kw., single-phase, air-blast transformers, and is delivered over four circuits to Montreal, where it is transformed into the city distribution systems.

The complexity of distribution at Montreal may be appreciated when it is noted that the current is of three different frequencies, namely, at 30 cycles from Shawenegan, at 60 from Lachine, and at 66 from Chambly, to which must be added the ordinary conversions to D. C., and to various distributing voltages, A. C.

The total distribution of power in the city from the various stations of the company varies from, say, 30 000 h.p. in the summer to as much as 53 000 h.p. at peak load in the winter, and this amount will be supplemented soon by delivery from the Soulanges station now under construction.

St. Lawrence River Branches.—The St. Lawrence River Basin is traversed by many large tributaries which will be discussed briefly in respect to such developments as have been carried out, but it may be stated briefly that, although many small towns and mining and milling centers have hydraulic and electric stations of modest proportions satisfying present needs, there yet remains opportunity for developing

millions of horse-power on such rivers as the Ottawa and its tributaries, the St. Maurice, the Saugenay, the Trent, Spanish, Nepigon, etc., and projects innumerable of greater or less merit are being studied carefully by engineers and financiers with a view to meeting industrial needs as they arise, and creating new centers and industries to consume the available power.

The Kaministiqua Power Company has constructed a generating station at Kakabeka Falls, on the Kaministiqua River, and transmits power to the City of Fort William (population 15 000) at the head of Lake Superior, where grain elevators, flour mills, foundries, and municipal utilities consume some 6 000 or 7 000 h.p. at the present time.

The development has a hydraulic head of 175 ft., and carries the water through two 10-ft. concrete tubes a distance of 6 500 ft. to a compensating reservoir, and thence by steel pipes to the power-house. The equipment consists of two 4 000 kw., 3-phase, 60-cycle, 4 000-volt units, driven by German-built turbines, and the current thus generated is raised to 25 000 volts by two banks of single-phase, air-cooled transformers of 1 500 kw. per phase; it is transmitted 19 miles over two circuits of No. 00 copper on wooden poles to Fort William.

In 1904 the Canadian Copper Company (otherwise the International Nickel Company) constructed a generating station on the Spanish River at Turbine, from which current is delivered to its mines and smelters at Copper Cliff near Sudbury, which lies north of Georgian Bay.

The construction of a large head reservoir created good operating conditions, and increased the hydraulic head to 85 ft. Two 2 000-kw. units are installed and a third is now under construction. The current as generated is 3-phase, 25-cycle, at 2 400 volts, and is stepped up to 35 000 volts for transmission. It is carried 29 miles to Copper Cliff, by two 3-phase circuits of No. 1 copper mounted on a double-pole line.

The company has various sub-stations where the current is distributed for service in the mines and at the smelters, which it is understood turn out more than one-half of the world's consumption of nickel.

The Mond Nickel Company is also installing a 1 500-h.p. hydro-electric station for similar purposes on the Vermilion River, and the current is to be transmitted 11 miles to the Victoria mines and smelters, 25 miles southwest of Sudbury.

The Town of Orillia (population 6 000) has constructed a power-station on the Severn River, and transmits some 1 600 to 1 800 h.p., at 22 000 volts, a distance of 20 miles. It distributes the current for industries as well as for pumping and lighting; and, by offering low power rates, has established a stable industrial center.

The City of Ottawa (population 75 000) is in the midst of many developed and undeveloped water-powers, and the latter, in the event of the construction of the Georgian Bay Ship Canal, would become readily available at low investment costs. In addition to the Ottawa River itself, many of its larger branches, especially in the Province of Quebec, are very great latent sources of hydraulic power, and it may be stated conservatively that within a radius of 75 miles of Ottawa City there can be developed on the Ottawa River and its main tributaries 500 000 h.p. at low water, which amount would be increased materially by the construction of the ship canal.

Near or in the City of Ottawa there are ten hydraulic and hydro-electric developments operating under various heads, but chiefly on the Chaudière Falls at from 22 to 35 ft. These developments form the industrial center of the city, and operate very extensive pulp and paper mills on both sides of the river. There are also cement mills, carbide furnaces, and other industries, and power for street railways, lighting, and general city use is also produced. About 35 000 h.p. is now developed, and, as many conflicting interests were contending for their respective shares of the flow of the river, it became evident that some improvement was imperative, so that an agreement to construct a proper dam at Chaudière Falls has been entered into and the work is in progress.

This dam will increase the hydraulic head somewhat, and will drown out the rapids immediately adjoining, thereby lessening the very serious present difficulties in respect to operating in the winter season, when the water is full of frazil, and chiefly to conserve the last drop of water in low-water periods, thereby increasing the available power at such seasons. However, in the near future, more power will be required, and it is proposed that the city itself develop the splendid powers quite near it on the Gatineau River, where a head of more than 80 ft. can be secured at a point within 8 miles of the city, and where 50 000 h.p. can be developed at one location. Considering the vast timber and mineral resources tributary to Ottawa, and its position as

Capital of the Dominion, it may be considered certain that the availability of unlimited hydro-electric power assures the industrial future of the city.

Quebec.—The Ottawa is only one of many great rivers flowing into the St. Lawrence from the north between Montreal and the sea, and, as all these rivers rise in unbroken forests on the Laurentian highlands, and are all well supplied with splendid storage facilities, they are all valuable sources of power. However, the sparse population has been a deterrent, and the only large works north of the St. Lawrence, and devoted to the generation of electric energy for transmission purposes, is on the St. Maurice River at Shawenegan Falls, where the Shawenegan Water and Power Company has created a large industrial town and also generates a large amount of power for transmission to Montreal, Sorel, Three Rivers, Thetford, etc., and will doubtless soon enter the field at Quebec in opposition to the present companies.

The hydraulic conditions at this place are most favorable, as the head-works consist of a natural lake, which prevents trouble from frazil in the winter. The drainage area is some 18 000 sq. miles, and a flow of 10 000 cu. ft. per sec. is considered the minimum. This, at a hydraulic head of 130 ft., provides more than 100 000 e.h.p. at all seasons. The works have been carried on partly for the generation of electric energy and partly to operate local industries hydraulically. The products of these industries are pulp, paper, aluminum, carbide, etc.

The electric station has three 4 000-kw. and two 7 000-kw. units, generating 2-phase, 30-cycle current at 2 200 volts, and this is transformed to 50 000 volts by banks of 1 100-kw. and 2 200-kw. single-phase, water-cooled transformers having a total capacity of 32 000 kw.

The transmission circuits are of aluminum cable on wooden poles. The Montreal service (85 miles) has been in operation for the past 10 or 12 years, and has given good satisfaction.

The delivery in Montreal is by two circuits, and amounts to 16 000 h.p.; besides which, the company serves the asbestos and other mineral districts on the south side of the St. Lawrence at Danville, Thetford, etc. The accomplishments of this company within the last ten years have been considerable; indeed, its works form the basis of many large industries, and it can be confidently expected that the whole 100 000 h.p. available at this one water-power will be made use of in the near future.

While Shawenegan is an instance of the water-power being developed as a foundation for industries and to supply distant markets, the developments in the vicinity of Quebec City (population 90 000) have been made strictly to supply existing needs. The installation on the Chaudière River, south of the St. Lawrence, has a hydraulic head of 114 ft., and is laid out for 6 000 h.p., but the equipment, thus far, is for not more than one-half this amount. The current is 3-phase, 66-cycle, and is generated at 11 000 volts. It is transmitted to Levis and also to Quebec City by insulated cable laid on the bed of the river.

Two developments north of the St. Lawrence, on the Montmorency River and on the Jacques Cartier River, supply current for city lighting, street railway operation, cotton mills, etc.

The Montmorency works, having a capacity of 4 000 h.p., have five units operating under a head of 220 ft., and generate 2-phase current at 5 000 volts, which is transmitted direct over four circuits to the city sub-stations.

The Jacques Cartier works have 2 000 h.p. installed, and the current is transmitted 18 miles at 20 000 volts over two 3-phase circuits to the city limits, where the company has a steam auxiliary station of about 2 000 h.p. to carry peak loads.

Some Conclusions.

A careful consideration of the electrical data contained herein will indicate the trend of activity in Canada in hydro-electric investments and the reasons therefor. In general, it may be said that on the Atlantic Coast and over the western and central portions of the great plains very little benefit is to be derived from such constructions, but over all the Provinces of New Brunswick, Quebec, Ontario, Eastern Manitoba, Western Alberta, and British Columbia, the continued construction of additional hydro-electric stations and the distribution of electric current to the practical limits of transmission may be confidently predicted. The resources of Canada, in minerals, timber, and agriculture, are so varied and so immense that the growth of industry as a legitimate outcome of the development of these resources, and the corresponding increase in population, may be looked on as a foregone conclusion, and it is as certain that, in this industrial growth, low-priced power will form a large factor.

At present the hydro-electric developments serve to a greater or

less extent such industrial centers as Montreal, Quebec, Ottawa, Peterboro, Toronto, Hamilton, Winnipeg, and Vancouver, besides a great number of smaller ones; but only a beginning has been made. Probably the 500 000 h.p. now hydraulically developed would include only stations of 1 000 h.p. or more, and there are literally many millions of horse-power available for development within easy reach of the present population; but, aside from the increased industrial demand, there are specific demands of great importance. Already, as noted, several mining interests are served by hydro-electric power, and with the growth of mining investments the importance of such service will increase. Then again, pulp and paper (with the depletion of United States supplies) will become a much more important industry than at present, and consequently much hydraulic and electric power for 24-hour service will be thus consumed.

Particular attention, however, is directed toward the enormous opportunities for electro-chemical industry. At Niagara Falls, N. Y., a large group of such industries is now well established, and in Canada a modest beginning has been made at Shawenegan, Ottawa, and Niagara Falls, Ont.; but Canada has in its great rivers sources of power as cheap as any in the world, except that the labor item may increase slightly the investment per horse-power beyond that of a similar construction in Europe. The whole question rests on transportation and markets; the latter, artifically restricted by the United States tariff, must seek an outlet in Great Britain, and as it is well demonstrated that $10 per horse-power-year, continuous power, at the low-tension switch-boards, is a remunerative price for large installations, and will cover all fixed charges, depreciation, and attendance, there would appear to be a good field for investment.

The financial condition in Canada at present, as regards hydro-electric enterprises, is worth considering. Some 10 or 15 years ago many of the brighter minds in the country seized the idea that, by securing valuable long-term franchises, on the one hand, for electrical service, and on the other for water privileges on near-by rivers, a splendid opportunity existed for profitable exploiting.

The net result at present is that, in almost all the large towns and cities, these services are administered by companies whose charges for power and light are only regulated by two things: either the exigency of the occasion, or the cost of a similar service produced by the

customer from coal usually imported from the United States; and as a good quality, run-of-mine, bituminous coal costs from $3.50 to $4.00 per ton in Ontario and Quebec and $6.50 per ton in Manitoba, it is evident that such competition is harmless, and the electrical companies were and are able to do business at very profitable rates.

In recent years, however, the municipalities have become active, and several smaller towns and cities have constructed municipal hydro-electric stations. The movement is very active at present in the Province of Ontario, and to a less extent in other provinces.

The struggle is bound to be prolonged and bitter; indeed, much bad feeling and resort to disreputable tactics have already been exhibited. Municipal and provincial credit has been attacked in the London market, and all the influence which heavy vested interests can exert is and will be directed against further municipal action.

The ultimate result is evident; the public mind will become more and more inflamed by the obstruction of the electrical companies, and, undoubtedly, in the end, the municipalities will carry out the expressed wish of the majority; but in the meantime public credit is unsettled, the manufacturers are playing a waiting game, and no one is benefited because the expansion of the market, and consequent advantage to every one, including the electrical manufacturing companies, has not taken place.

It would appear advisable to consider the appointment of a Public Service Commission by the Dominion Government to regulate the rates which may be charged by companies already in operation, so as to secure that benefit to the public which the presence of abundant water-powers warrants, because it is not probable that, at the most, more than a majority of the industrial centers will be served municipally, and, judging by the results obtained by the Dominion Railway Commission, much satisfaction might be obtained and the public mind be set at rest if a rate-regulating commission for electric utilities, with wide powers of action, should be appointed.

Concurrently, however, with the distribution of electric power for utilization in cities, etc., it is to be anticipated that very large constructions will be undertaken in the near future to furnish power for the operation of railways over heavy grades, where coal is expensive or traffic dense, and that at the same time the natural resources of the country will develop to an increasing extent, and that

in mining work, pulp and paper manufacture, electric smelting, and general electro-chemical service will be found the most attractive fields for the enormous and not fully appreciated sources of cheap and widely-distributed power which is now wasting in so many districts of Canada.

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PAPERS AND DISCUSSIONS

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SAMPITTIC SURFACING.

Discussion.*

BY MESSRS. L. W. PAGE, PREVOST HUBBARD, ARTHUR H. BLANCHARD, PHILIP P. SHARPLES, H. W. CLARK, C. E. GRUNSKY, AND HENRY B. DROWNE.

Mr. Page. L. W. PAGE, ESQ.† (by letter).—While the writer is in hearty accord with the growing desire exhibited by road engineers to obtain standard specifications for bituminous cements to be used in road treatment, and while he realizes to the fullest extent the benefits which would be derived from such specifications could they be produced, there are certain points which are apt to be overlooked by the engineer and should receive serious consideration.

In the first place, local conditions, such as climate, location of the road, quantity and character of the traffic to which the road is subjected, as well as the character of the road material, have such an important bearing on the requisite characteristics of a bituminous binder that even when the same methods of construction and application are followed, it is doubtful if any one cement will prove equally satisfactory for all cases. In many instances experiment alone will determine what binder is best suited to meet given conditions, and the impracticability of following the same strict specifications for all cases, therefore, is apparent.

Broad specifications in regard to the elimination of certain objectionable properties which have invariably produced bad results,

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† Director of Public Roads, Washington, D. C.

Mr. Page.

and favoring those characteristics which are essentially necessary to a good binding material, can be framed, of course, but strict limitations with respect to certain constituents, such as free carbon, fixed carbon, paraffin contents, etc., are, at the present time, based on matters of personal opinion rather than on established facts. This is particularly true of country roads, as considered apart from city pavements, and is largely due to two facts: (1) That, owing to necessarily less refined methods of construction and less severe traffic conditions, binders which would be of little service for city work will often prove satisfactory for country highway work; (2) that, at the present time, there are but few reliable data as to the characteristics of the binders which have been used in experimental work, and their effect on the results obtained.

From a careful consideration of this subject the writer is led to the conclusion that the framing of specifications for bituminous road binders is, and will be for many years to come, a matter of expert opinion for individual cases, and that, in view of the fact that the near future will see an immense amount of bituminous road work carried on, every progressive road engineer should make a study of bituminous road binders, and, in all reports of experimental work, should include the results of an examination of the binder which he has used. By so doing, any binder which has met his requirements in a satisfactory manner can be identified, and specifications can be made which will enable him to obtain a like material again and duplicate his results. His experience will also be of considerable value to others who are working along similar lines.

In regard to Mr. Crosby's proposed pitch compound, it might be said that, while there is little doubt that under certain conditions a material meeting his specifications would prove satisfactory, it is questionable whether such a guaranteed material could be generally obtained at anything like a reasonable cost. The time required to make an examination covering these specifications is another objectionable feature. It might be added that, while Section 287 states that "the pitch compound to be used shall be a hydrocarbon, refined from tar or petroleum, or made from natural asphaltic pitches by the use of a proper solvent or flux", according to the writer's experience practically all gas-house tar pitches are eliminated by Clause *a*, and even coke-oven and water-gas tar pitches by Clause *j*.

According to the writer's belief, any ductility test made as described in Clause *e* is of little or no service in determining the road-binding value of the original material, for the reason that, when subjected to temperatures sufficiently high to give a residue of the consistency stated, many oils are so altered that the residue thus artificially produced in no way represents the binding value of the original material.

Mr. Hubbard.

PREVOST HUBBARD, ESQ.* (by letter).—Before discussing Mr. Crosby's specifications for a basic pitch compound, the writer wishes to state that his views relative to the practicability, or rather the impracticability, of framing strict specifications which will meet all conditions satisfactorily are in close agreement with those presented by Mr. Page.

Considered from a chemical standpoint alone, the wide differences in composition existing between the two classes of bituminous binders —tars and asphaltic oils—make it a difficult matter to deal with both in one set of specifications. This can readily be shown by taking up the various clauses separately and determining their value as applied to each class of binders. By so doing, a number of interesting facts pointing toward the most rational methods of examination for each class may also be developed. As a consideration of this subject in detail, however, would constitute a paper by itself, only those points which the writer believes to be most important will be included in the following brief discussion.

Clause a.—From the examination of a considerable variety of bituminous road binders in the Laboratory of the United States Office of Public Roads, it would seem that any specification limiting the percentage of free carbon to 10 is applicable to tars only. A total bitumen clause could be substituted for oils, and while the two determinations are made in much the same way, the amount of material insoluble in carbon bisulphide could be limited to 2% for most asphaltic oil products. In cases where a material such as Trinidad asphalt had been fluxed with an oil to produce the pitch compound, however, allowance would have to be made for a considerable amount of inorganic material which, of course, would be insoluble in carbon bisulphide. As applied to tars, a clause limiting the free carbon contents to 10% would bar out practically all gas-house coal tars, and, while the writer firmly believes in the superiority of low free carbon tars, it would seem hardly fair to eliminate those containing more than 10%, as this class constitutes a large proportion of the tars produced in the United States, and, in some instances, good results have attended their use in road construction.

Clause b.—A determination of fixed carbon in a bituminous road binder is of value mainly for the purpose of determining the mechanical stability of the bitumens present. As this property, within certain limits, is believed to be almost directly proportional to the percentage of fixed carbon, it would seem that if any limitation is made it should be in the other direction, that is, the lowest limit should be stated rather than the highest. When applied to the bitumen itself, it is extremely doubtful if any ordinary pitch compound would show fixed carbon as high as 20%, and, if it showed as high as 15%, it could

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hardly be expected to meet the requirements of Clause *g*. In addition, it might be said that while an oil or asphaltic product with fixed carbon running as high as 7% might be satisfactory for construction work, the same would not necessarily be true of a tar pitch, even if the free carbon content were not considered. A fixed-carbon specification, therefore, would seem to be of value mainly as applied to oil and asphaltic products, in the manner indicated. Mr. Hubbard.

Clause c.—As there is considerable difference of opinion among chemists, at the present time, as to the value of a paraffin determination applied to road oils, the writer will not consider it in detail. It might be stated, however, that, without question, such a determination is unnecessary in the examination of tars. The writer also believes that, as a rule, it is unnecessary in the examination of oils, as the characteristics of the base obtained from a volatilization test will show whether paraffin hydrocarbons are present in sufficient quantity to injure the material for road use.

Clause d.—The main objection to this clause is that a 24-hour continuous volatilization test at a temperature of 105° cent., is impracticable for most laboratories.

Clause e.—While no criticism of a ductility test, if it can be made on the original material, is here intended, it would seem to be of little or no value as applied to residues produced at high temperatures, for the reason that in producing such residues many bituminous materials are chemically altered in such a way that a ductility test made in the manner described cannot be depended on to represent the characteristics of the original material. This can often be shown by a comparison of the relative fixed carbon, naphtha insoluble, and carbon tetrachloride insoluble determinations made upon the original materials and the residue.

Clause f.—This clause will not be discussed, owing to its direct relation to Clause *d*, which has already been considered.

Clauses g, h, and i.—A determination of viscosity and melting point is undoubtedly of considerable value when it is desired to apply a material in a given manner and under given conditions; but the writer believes that a determination of the consistency of a residue resulting from a volatilization test made at comparatively low temperatures, say, 163° or 205° cent., is of more service than that of the percentage of residue of given consistency which can be produced at high temperatures. The reasons for this are the same as those mentioned under Clause *e*.

Clause j.—While this specification might be applied to oil products, it would bar out practically all tar preparations which, as a rule, contain considerably more than 20% of oils which can be removed by distilling between 170° and 270° cent.

Clause k.—The only criticisms applicable to this clause have been made evident in the discussion of Clause *a*.

Mr. Hubbard.

Clauses l and m.—These clauses would seem to be applicable and necessary to any set of specifications, but it should be remembered that under ordinary conditions where a carload of material is to be held up until an examination can be made, a very limited time can be allowed for such an examination. From this point of view, alone, it is evident that an examination, according to Mr. Crosby's proposed specifications, requires entirely too much time to be practicable.

Conclusions.—In general it would seem inadvisable to attempt to frame specifications for bituminous road binders, except for individual cases. Somewhat different methods of examination are required for asphaltic oil products and tar products. Owing to the fact that no standard methods of examination have as yet been established for these materials, reports from different chemists upon the same material are apt to differ greatly. The establishment of standard methods of examination, therefore, should receive first consideration; otherwise, exact descriptions of the methods which should be followed will have to be included in each set of specifications. Owing to lack of detailed data as to the exact effect of composition upon the results obtained in practice, an examination of bituminous road binders is mainly of value as a means of identification and for the purpose of determining which of a number of materials is best adapted to meet given conditions. Strict specifications, therefore, cannot be made satisfactorily until such data are available. In future experimental work with bituminous road binders, therefore, it is recommended that the results of an examination of the binder be included in any report which may be made.

Mr. Blanchard.

ARTHUR H. BLANCHARD, M. AM. SOC. C. E.—Highway engineers owe Mr. Crosby a debt of gratitude for presenting for criticism, tentative specifications covering pitch compounds, and thus precipitating a discussion of bituminous materials in its entirety.

The present status of specifications covering bituminous materials is unsatisfactory from the standpoint of both the producer and the purchaser, and is due primarily to a lack of knowledge of the methods of specifying the requisite properties, and to the insufficiency of published data relative to the results attained by using various bituminous materials in divers ways.

Without doubt, every highway engineer dealing with the construction and maintenance of roads subjected to excessive motor-car traffic will welcome enthusiastically the adoption of standard specifications which will permit of calling for bids for a bituminous binder, with the expectation that the material submitted will be adapted to the exigency. The attainment of this desideratum will take from two to five years of co-operative work on the part of chemists and of the highway engineers who have charge of the construction and maintenance of bituminous surfaces.

Mr. Blanchard.

One method of procedure which would result in the acquisition of the desired data in the near future is as follows: First, keep full notes during construction, covering the points mentioned below; second, make a complete chemical and physical analysis of the bituminous material in accordance with standard methods; and third, keep a complete record of the condition of the road under varying climatic conditions. By compilation and a comparison of the data acquired it will be possible, finally, to determine the essential requisites for bituminous material to be used in any particular method of construction, and the requirements which must be included in specifications in order to secure materials which possess these properties.

The essential data to be recorded covering the construction of bituminous surfaces are as follows:

ROAD PRESERVATION.

Data Relative to Construction.—

State
County.
Town.
Name of highway.
Class of highway.*
Nature of traffic.*
Traffic census.†
Distance to nearest railroad station.
Distance to source of materials.
Length.
Width.
Area, in square yards.
Nature of subgrade.
Maximum depth and length of fill.
Maximum, mean, and minimum grades.
Kind of stone, gravel or sand used.
Sizes or grades of broken stone or gravel.
Contractor.
Dates when construction was commenced and completed, or months during which construction was carried on.
Temperatures at 8 A. M. and 2 P. M. on each day during construction.
Climatic conditions on working days.
Location of car tracks, if any, relative to bituminous surface.
Road exposed or protected.
Number of hours in legal working day.
Average rate of progress per day, in square yards.
Specifications covering construction of bituminous surface.
Details of construction.
Data relative to contract covering the construction of the road complete.
Bituminous material furnished by contractor, or by party of the first part.

* See *Transactions*, Am. Soc. C. E., Vol. LXI, p. 446.
† See *Engineering News*, Vol. 61, p. 466.

Mr. Blanchard.

Cost of bituminous material per square yard, if furnished by party of the first part.
Cost per square yard complete.
Estimated cost per square yard over and above cost of similar construction minus bituminous material.
Data relative to contract covering the construction of the bituminous surface as a secondary contract.
Lump sum contract, or price per square yard.
Bituminous material furnished by contractor, or by party of the first part.
Cost of bituminous material per square yard, if furnished by party of the first part.
Cost per square yard of bituminous surface complete.
Day labor plus percentage contract.
Bituminous material furnished by contractor, or by party of the first part.
Cost of bituminous material per square yard, if furnished by party of the first part.
Cost per square yard of bituminous surface, complete.
Rebate on watering cart, labor, screenings, etc.
Percentage allowed on extra work.
Unit prices (hour basis).
Common laborer.
Foreman.
Man capable of handling bituminous material.
Double team.
Single team.
Rental of mixing plant, kettles, and accessories, distributor, or other special apparatus.
Roller.
Watering cart.
Scarifier.

Every effort should be made within the next two months to standardize methods of determining chemical and physical properties, in order to obtain the maximum amount of information from the construction work of 1909.

In the laboratory of the State Board of Public Roads of Rhode Island, tests are made to determine the following properties in accordance with methods adopted by Mr. Crosby and the writer:

Tars.—
Water-soluble materials.
Specific gravity.
Free carbon.
Fixed carbon.
Loss by evaporation for 21 hours at 105° cent.
Penetration of residue.
Melting point of residue.
Loss by distillation.
Viscosity.

Mr. Blanchard.

Asphalts and Asphaltic Oils.—
Melting point of solid asphalts.
Penetration of solid asphalts.
Water-soluble materials.
Specific gravity.
Free carbon.
Fixed carbon.
Paraffin.
Loss by evaporation for 7 hours at 170° cent.
Loss by evaporation for 21 hours at 105° cent.
Penetration of residue.
Melting point of residue.
Loss by evaporation for 21 hours at 225° cent.
Penetration of residue.
Melting point of residue.
Solubility in 88° Baumé naphtha.
Viscosity.

Other tests which are made in the Rhode Island laboratory are for the determination of the cohesive strength of the material under gradually applied tension, and the cohesive strength under impact.

The machines used in the Rhode Island laboratory for testing the cohesive strength of bituminous materials were designed by Henry B. Drowne, Assoc. M. Am. Soc. C. E., Assistant Engineer, State Board of Public Roads, and he will describe these machines and the methods of conducting the tests.

The object of the cohesion tests is to acquire information relative to the comparative value of various bituminous materials from the standpoint of their ability to resist: first, the action of motor-car traffic on the surface layer; second, the impact of horses' hoofs; and finally, to devise specifications covering these essential properties.

In writing specifications for bituminous materials to be used in highway construction, two facts must be kept constantly in mind, if such specifications are to prove satisfactory: first, the relationship between a given requirement and the value of that property of the material when considered from the standpoint of a road binder; and second, the advisability of limiting the requirements of the specifications to those for which the tests can be made in a reasonable period.

Mr. Sharples.

PHILIP P. SHARPLES, ESQ.*—In discussing the subject of tests for bituminous materials, a sharp line should be drawn between methods of analysis suitable to be inserted in a specification and those which are of scientific or academic interest. Both the consumer and the manufacturer desire that a product with certain characteristics should be defined. The tests which shall confine the product within the limits should be as simple as possible and easily executed in any works laboratory without special apparatus.

* Chemist, Barrett Manufacturing Company, Boston, Mass.

Mr. Sharples. To confine this discussion to a criticism of the tests for tar materials, the specification, as set forth by Mr. Crosby in its amended form, requires far too many tests, and some of them are much more complicated than necessary. Mr. Smith has already outlined the objections to the tests, and Mr. Hubbard's discussion is along the same lines.

Mr. Crosby's specifications call for fifteen different tests. A specification calling for not more than six or seven tests should define the product with sufficient clearness. As a model, in the direction of simplicity, the specification of the Massachusetts Highway Commission for the recently advertised Milford work is to be commended for its form, although the figures are subject to modification. Mr. Clark discusses the Massachusetts specification, and a few comments are offered.

In the case of Mr. Crosby's tests, as Mr. Smith has pointed out, the free carbon content required is too low. A definite minimum and maximum should be set. The test for evaporation loss should be modified so that it may be compassed in one working day. Equally good results may be obtained, it is believed, by shortening the time and raising the temperature slightly.

The viscosity test should be made on a standard instrument, and preferably should be modified so that it will indicate the viscosity at ordinary temperatures.

To conclude, the speaker believes that the following tests are sufficient to define a coal-tar pitch compound: Specific gravity; free carbon; distillation test, including tests of oils and pitches obtained. To these may be added a viscosity or penetrometer test.

Mr. Clark. H. W. Clark, Esq.*—The speaker has been very much interested in Mr. Crosby's paper, and is glad to take part in the discussion. As has been stated by the author, the paper differs, in many respects, from the form in which the speaker first saw it, that is, as now presented, the limiting figures in the specifications have been omitted.

During the past nine months the Highway Commission of Massachusetts has had under way many laboratory and road experiments on this subject, and a large number of samples have been examined chemically and submitted to physical tests. Up to the present time, about 250 analyses of tars, refined tars, asphalt oils, oil asphalts, trade compounds, etc., have been made, and, as the result of all this work, certain specifications have been drawn up in regard to tests which these road materials should pass to show certain qualities which they must possess in order to make them of value in road work. In this experimental work and in drawing up the specifications the Commission has had in mind no theoretical material which, perhaps, might be of greater value as a binder than any now on the market, such a

* Chemist, Massachusetts State Board of Health.

material as probably would have to be found if all Mr. Crosby's specifications, as first presented, were held to; but, rather, an endeavor has been made to find out, from experimental roads and analyses of materials, which of these binding materials in each class has thus far, apparently, given the best results, and the specifications have been framed so that only the best of these materials, judging from chemical analyses and practical results, will pass them. It has been thought necessary and most practical to have separate specifications for refined tars, asphalt oils, oil asphalts, and natural asphalts, and in compiling these specifications the aim has been to make them as simple as possible, that is, not to make the labor of analytical chemical work and physical tests in various laboratories unduly arduous, but to include such tests as will surely, from what is known at present at least, cause the use of desirable binders. Mr. Clark.

The methods of analysis used by the Massachusetts Highway Commission differ to some extent from those outlined by Mr. Crosby. They are simpler, and it is intended to make them public soon, in order that those submitting binding materials can know just how the laboratory work of the Commission is done. It is not necessary to enter into a discussion of these methods, however, although attention may be called to one or two points. Considerable has been said in regard to physical tests, especially in the way of suggesting that tests similar to those to which cement is subjected may be eventually developed. It has been believed from the beginning that some work of this sort was necessary, and, for the past nine months, briquettes have been made of certain proportions of sand and the bituminous materials undergoing examination, and these briquettes have been submitted to certain tests. This was started coincidently with the chemical work, as it was believed that materials proposed as binders should show some binding property, and this property can certainly be shown more quickly by physical than by chemical methods. Of course, the chemical analyses are of more importance in many respects than any physical test. The specifications which the Highway Commission has just issued, include a briquette test. In studying these briquettes, however, it is fairly evident that it does not necessarily follow that those binders which soonest produce hard and strong briquettes are the most valuable. In Mr. Crosby's paper, as first presented to the speaker, there was one specification covering the loss which the material should undergo when 20 grammes were heated in a flat-bottomed dish, etc., for a certain number of hours. In its work the Massachusetts Highway Commission has found that the various classes of materials must be differentiated in this respect, for the refined tars and asphalt oils act very differently one from another in this question of loss of weight, and that, on the other hand, the oil asphalts may, and often do, gain in weight when kept at a high temperature for a prolonged period.

Mr. Clark. The type of viscosimeter used is believed to be practical and simple, but no hard-and-fast rule can be given in regard to the absolute time which a good binder must take in passing through a capillary tube of a certain diameter, equally good binders, as far as has been discovered, showing great variation in this respect, and in the specifications for each class of material quite wide limits of viscosity have been allowed.

Mr. Crosby appears to have made no statement in regard to specific gravity, but it has been found that the best materials of the different classes should have a certain specific gravity, if they are to be of value. For instance, the specifications of the Massachusetts Highway Commission state that the asphalt oils must have a specific gravity of 0.97. Oils with a specific gravity much less than this are light oils, with very little binding power and very little bituminous residue. They may be of some value in treating old roads, perhaps, but they are probably of little value in construction. To illustrate the simplicity of the specifications of the Massachusetts Highway Commission, those for refined tar and asphalt oils are included here. As stated previously, however, it is not to be supposed that the specifications cannot be improved, and that they will not be improved by the Commission eventually, but it is safe to say that any refined tar answering to this set of specifications will be included, as nearly as can be determined, in that class which gives the best results in road work.

Suggested Specifications for Refined Tar.

The tar shall be uniform in color, character, appearance, and viscosity, and shall have the following qualities:

1.—It shall contain not more than 0.5% of mineral matter or dirt.

2.—It shall have a specific gravity between 1.18 and 1.25.

3.—It shall not contain more than 14% by weight of free carbon.

4.—It shall contain no body which distils at a temperature lower than 225° cent.; not more than 10% by weight shall distil below 270° cent., and it shall contain at least 65% by weight of pitch or bituminous material remaining after all bodies up to 360° cent. have been distilled.

5.—When 20 grammes are heated in a flat-bottomed dish, 3 in. in diameter, for twenty-one hours, in an oven kept at a temperature of 100° cent., the loss shall be not more than 10% by weight.

6.—It shall be of such viscosity that 60 cu. cm., measured at room temperature (78° Fahr. or 26° cent.), shall, when at 100° cent., be not less than 85 sec. and not more than 240 sec. in passing through a viscosimeter orifice $\frac{5}{64}$ in. in diameter, when acting under a head of 4½ in.

7.—When 12½% by weight of the material is mixed with 87½%

Mr. Clark.

by weight of sand, of such grade that all of it will pass through a sieve having 10 meshes to the linear inch, and that practically none of it will pass through one with 190 meshes to the linear inch, and briquettes, 3 in. square and $\frac{1}{2}$ in. thick, are made, such briquettes will harden to such an extent in seven days, at ordinary room temperature, that when they are laid flat and held by their edges by two parallel knife-edge bars, they shall not bend when a weight is suspended from a third knife-edge or parallel bar placed across their center, until this weight reaches 200 grammes, and they shall not break at less than 250 grammes, and the weight which causes bending shall not be greater than 80% of the weight which causes breaking.

SUGGESTED SPECIFICATIONS FOR ASPHALT OILS.

The oil submitted shall be of uniform color, appearance, general character, and viscosity. It shall contain no bodies not naturally present in an asphaltic oil, and shall fulfill the following requirements:

1.—It shall not contain more than 0.5% of dirt or adventitious mineral matter.

2.—It shall have a specific gravity of at least 0.97.

3.—It shall not contain more than 1% of matter insoluble in carbon bisulphide, and shall not contain more than 10.0% of matter insoluble in petroleum ether.

4.—It shall contain no body which distils at a temperature lower than 250° cent., and shall not lose more than 55% by weight by distilling to 360° cent.

5.—It shall be of such viscosity that 60 cu. cm., measured at room temperature (78° Fahr. or 26° cent.), shall, when at 100° cent., be not less than 5 min. nor more than 10 min. in passing through a viscosimeter orifice $\frac{5}{64}$ in. in diameter, when acting under a head of $4\frac{1}{4}$ in.

6.—When 20 grammes are heated in a flat-bottomed dish, 3 in. in diameter, for twenty-one hours, in an oven kept at a temperature of 100° cent., it shall not lose more than 5% by weight.

7.—When $12\frac{1}{2}$% by weight of material is mixed with $87\frac{1}{2}$% by weight of sand, and briquettes 3 in. square and $\frac{1}{2}$ in. thick, are made, these briquettes shall keep their shape and show some bond.

Mr. Blanchard's remarks in regard to mixtures of tars or refined tars and asphalts are very interesting. The fact that he builds the beautiful roads, which everybody seems to testify to, with this mixture, is simply another illustration that theory and practice do not always

Mr. Clark. agree. Theoretically, Mr. Blanchard's roads should not be so perfect with this mixture, as the two bodies, when mixed, do not combine, as can be seen easily by examination under a microscope, and if the proportion is not right, a spongy, crumbling mass results, and briquettes made of sand and this mixture have no cohesion whatever. The fact stated by Mr. Blanchard that the asphalt comes to the surface and forms a surface coating, simply shows that the two bodies are not combined, but simply mixed, and the asphalt separates itself from the tar. Judging from the statements made, it seems to be one of the ways, and, perhaps the best way, of getting a surface coating of asphalt.

Mr. Grunsky. C. E. GRUNSKY, M. AM. SOC. C. E. (by letter).—Prior to 1900 practically no attempt had been made to prescribe the quality of bituminous materials to be used for paving purposes in San Francisco, Cal. The first bituminous rock pavements (rock asphalt) were laid about 1887. The specifications were silent as to the physical or chemical properties of the asphaltic material, the most that was prescribed being a minimum percentage of bituminous material. Fortunately, the materials which were then available were admirably adapted for paving purposes; the sand impregnated with the asphalt was of good quality, well graded, and, as has been demonstrated under the test of actual use, the asphalt was of a quality in every way suitable for paving purposes. It is not surprising, therefore, that the paving contractors succeeded, by a judicious mixing at the quarries of materials of various degrees of hardness, in furnishing rock asphalt serviceable alike under the hot summer sun in the interior cities of California and under the cool breezes of the Bay City.

Notwithstanding several failures of asphaltic wearing surfaces at Sacramento, there was put down, in 1888, a pavement in the rear of the State Capitol, under the advice and supervision of the writer, which, after 20 years, is still in fine condition, and on which there have been no repairs. The traffic, however, is not heavy.

The generally satisfactory quality of the bituminous materials used in California in street and road work is well recognized. The following, condensed from reports which the writer made in 1903, referring to the bituminous rock pavements of San Francisco, bears upon this fact. Some of these pavements were already more than 15 years old. The aggregate length of streets paved with bituminous rock was then 107 miles, and this mileage included many down-town streets having heavy traffic. The experience, since the first of these pavements was put down in San Francisco, indicates that about $200 per mile per year, with an additional allowance for re-surfacing annually an area equal to one-tenth of the roadway, the wearing surface of which is more than 10 years old, would keep the streets in good repair. This was not the amount actually expended for repairs. Many streets were in a neglected condition. The actual appropriation for repairs had

fallen about $80 000 behind the amount that should have been expended, and the current appropriations for the purpose were only about one-half of what was required to keep the streets in good condition. Mr. Grunsky.

At that time it was estimated that pavements of this material, when more than 10 years old, would need minor repairs at a cost of about $1\frac{1}{8}$ cents per sq. yd. per annum, and that, in addition thereto, the wearing-surface renewal cost would be about $3\frac{5}{8}$ cents.

Several years prior to the date to which the above remarks refer, the City of San Francisco, under the writer's supervision, had established a testing laboratory. Plans were made for a careful examination of the asphaltic materials offered for use, as well as of asphalts extracted from pavements which had been in service for some time. It was hoped that such an examination would enable the chemist to detect those properties of bituminous material which were essential in making it valuable as a binder in the wearing surface of a street or road. No information is at hand to show whether or not this work has been carried forward successfully.

The abundance of bitumens and of oils with an asphalt base in California, which are admittedly serviceable in road-building, has rendered the use of coal-tar products out of the question in that State. Not even the by-products of oil distillation were willingly accepted for use on city streets. On this subject, in 1902, as City Engineer of San Francisco, the writer called attention, as he had done in previous official reports, to the desirability of permitting the general use of oil asphalt in paving work. The view was taken that California is a great producer of oil, and, therefore, as far as practicable, the home uses of the products of the oil industry should be demonstrated and extended. The serviceability of oil asphalt as a substitute for the natural solid asphalt was not determinable short of a practical test.

"Its use on a few of the city streets, under various conditions of traffic and on different kinds of foundation will, it is hoped, go far toward determining the question of serviceability. It may be stated in this connection that the California oils nearly all have an asphalt and not a paraffin base, and that, therefore, the possibility of producing an asphalt, in the course of the oil-refining process, that will have properties practically identical with the natural solid asphalts, must be recognized."

It was pointed out that the market was stocked with the products of many refineries, that the origin of the products was not always traceable, and that even when the refinery was known, the source of the oils and the character of their mixtures might remain in doubt. It was also stated that at least five years should elapse before final judgment can be passed on an asphaltic paving material, and that, in some instances, even 10 years might be required to reach a verdict.

In the meantime the use of oils, and particularly of the heavy oils of the California type, as a binder in sampittic wearing surfaces has

Mr. Grunsky. been rapidly extended, and the serviceability of California oils for such use has been demonstrated.

It is not to be hoped that there can be devised any laboratory process which will reveal fully the value of any untried bituminous material as a binder. This value will depend not only on a satisfactory condition of the road surface at the time of its completion, but also on its behavior under traffic, and this, in turn, will depend, among other things, on the nature of the rock particles which are cemented together, and mainly on the life of the bituminous binder. This useful life cannot be predicted for untried material until possibly after a sufficient number of successes and failures in actual practice have been recorded, together with adequate information relating to the properties of the asphaltic materials disclosed in the laboratory, and of all conditions under which such materials entered into the work. It may then be possible to write specifications so that they will exclude all undesirable materials while admitting all those which are known to be good. It appears to be hopeless at this time to devise laboratory experiments or tests of the quality of substances of this class which, with the information thus gained, apart from actual use, will determine their paving value. The admission of untried materials, therefore, will always be with more or less risk of failure. Moreover, it follows from this that laboratory tests prescribed by specifications should be simple and require but little time, and that no attempt need be made to imitate in the laboratory the conditions under which the materials are to come into use. The old method of determining the petrolene and asphaltene contents of the bituminous compound, and then determining the viscosity, ductility, adhesiveness, hardness, and behavior in water, and under temperature changes, of a distillation residuum is just as likely to lead to a form of reliable specifications for materials of this class as a more complicated examination.

While the materials which are offered are being tried out on roadways, however, full records of their chemical and physical properties should be preserved, and the methods of ascertaining these properties should be standardized, so that from year to year the results may be compared. This is a matter which should not be confounded with the prescribing of simple yet adequate laboratory tests to appear or to be referred to in specifications for road work. The thorough discussion of this paper will be welcomed by all highway engineers, and it is to be hoped that the author's efforts to secure a standardization of laboratory processes and of specifications will bring about the desired result.

California is in the movement for good roads. The State is soon to vote on a proposition to issue bonds to the amount of $18 000 000 for new roads. One county is now expending $3 000 000 on its roads, and, in another county, a bond issue of $1 800 000 has been voted within the last few months for a similar purpose.

FIG. 1.—CEMENT TESTING MACHINE, MODIFIED FOR MAKING COHESION TESTS.

FIG. 2.—IMPACT MACHINE.

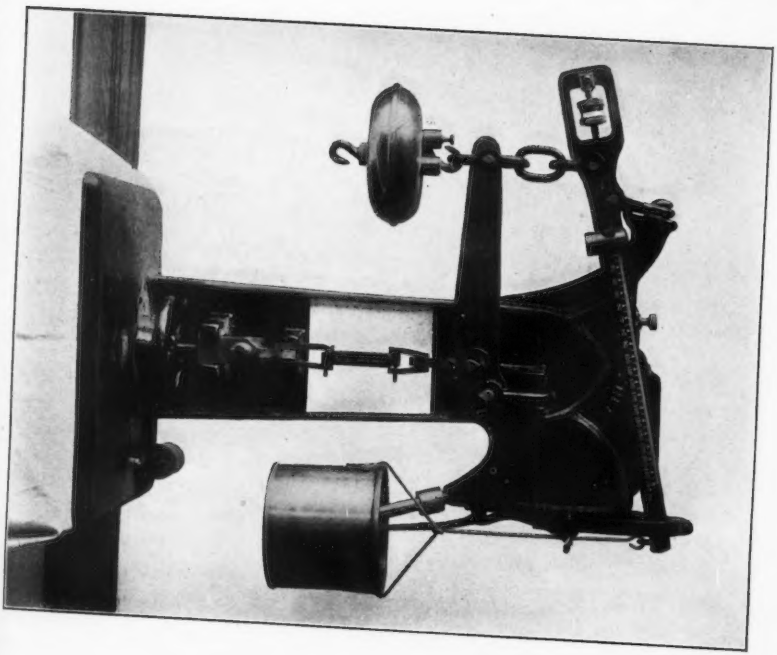

FIG. 1.—CEMENT TESTING MACHINE, MODIFIED FOR MAKING COHESION TESTS.

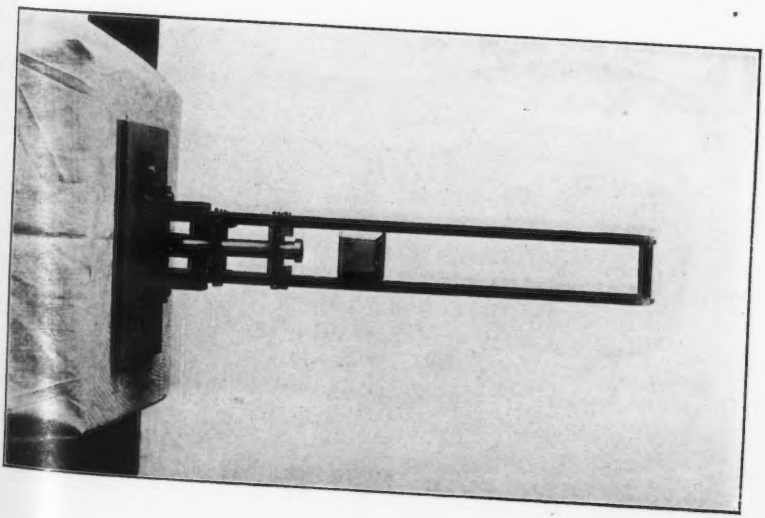

FIG. 2.—IMPACT MACHINE.

UNIV. OF MICH.

HENRY B. DROWNE, ASSOC. M. AM. SOC. C. E. (by letter).—It seems to the writer that at the present time no very rigid set of specifications as to the essential qualities of some of the bituminous materials now on the market can be written. Chemical and physical tests will show that a bituminous material possesses certain characteristics, but whether or not these characteristics are detrimental or beneficial can only be ascertained by observation of the action of the material in the road for a period of some years. It is important now, however, to develop tests by which the materials can be analyzed, and to have enough tests, and of such a character as to account for all phases of the action of the material, from the standpoint of both construction and service. In order that work in different parts of the country can be compared, it is of the utmost importance that uniform methods of testing bituminous materials should be adopted, and the writer believes that Mr. Crosby's paper and its discussion will aid in bringing about this result. Mr. Drowne.

In the Rhode Island laboratory, the engineers have been trying to develop two physical tests of bituminous materials to show the cohesiveness of the different materials under uniform tension and sudden shock. For convenience, the test with the steady pull is called the cohesion test, and that with a sudden shock, the impact test. A series of experiments was made on 1-in. cubes of stone cemented together with different bituminous products. The methods used in making the tests were rather crude, but the results showed that with different appliances a satisfactory test might be expected. After some study, it was decided to use metal blocks for the impact test and metal plates for the cohesion test. Metal was chosen because the same surfaces could more easily be produced, the metal is easily cleaned, and is durable; whereas, with stone, the kind of stone, its structure, and porosity would all be factors which might affect the results of the tests.

A Fairbanks cement testing machine, No. 3 194, is used to provide the uniform tension required in making the cohesion test. This machine was selected because many of the State highway and other laboratories are equipped with one. The modifications necessary to adapt this machine to the cohesion test are simple and readily duplicated, as is shown by Fig. 1, Plate XX. The two clamps which hold the cement briquettes are removed, together with the cast-iron "U's" by which they are attached to the knife-edges and pin. These fittings are replaced by bronze castings of a different and lighter pattern.* As these castings are lighter than the ones removed, sufficient weight must be added to the large ball to bring the scale beam to a true balance. The test plates are bronze castings of such a size that a

* Some of the illustrations for this discussion were not received in time to be reproduced in this number of *Proceedings*. They will be shown when the paper is printed in *Transactions*.

Mr. Drowne. surface of 2½ sq. in. can be obtained. A shoulder is provided on the plates, so that when they are hung in the testing machine there will be no eccentricity of pull. The plates are cemented together, and are then connected to the upper and lower castings and tested.

The impact machine is practically a miniature pile-driver, as shown by Fig. 2, Plate XX. The essential parts are the leads, the 2-lb. weight, the plunger and guides, the clamp to hold the test blocks, and the stops on the leads to catch the weight when the specimen fails. The test blocks are 1-in. bronze cubes. The blocks are cemented together and placed in the clamp so that one of the blocks comes under the plunger. The weight is then raised and allowed to drop on the plunger; the number of blows are noted, and also the height of fall of the weight that causes failure.

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AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

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THE DESIGN OF ELEVATED TANKS AND STAND-PIPES.

Discussion.*

By Messrs. Charles B. Burdick, O. E. Selby, and Charles C. Hopkins.

Mr. Burdick.

Charles B. Burdick, Assoc. M. Am. Soc. C. E. (by letter).—The author is to be commended for presenting in concise form the requirements for the design of elevated tanks and stand-pipes. The value of the paper, however, would be greatly increased by taking up the items separately and giving the reasons which led to the conclusions.

In regard to Section 7, on Unit Strains, apparently no especial consideration is given to the action of ice, which is generally regarded as producing an important stress. In a measure, this is allowed for in the moderate permissible tensile stresses in plates, and also in the requirement for minimum thickness of plates, which gives increased strength at the top of the tank where the ice action is probably at a maximum.

It seems to the writer, however, that the matter of climate should be given consideration in fixing allowable unit stresses. Good design will not permit the same strains in such extremes of climate as Florida and Minnesota. It is considered good practice in the vicinity of Chicago and northward to limit the plate unit stress to 10 000 lb. per sq. in. of net section for tanks in which the plate is directly exposed to the weather. It is believed, however, that proper design permits greater unit stresses where the steel is enclosed or encased by a protective covering which eliminates or reduces to a minimum the formation of ice.

* This discussion (of the paper by C. W. Birch-Nord, Assoc. M. Am. Soc. C. E., printed in *Proceedings* for March, 1909), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

Mr. Selby. O. E. SELBY, JUN. AM. SOC. C. E. (by letter).—This paper presents a desirable addition to the list of standard engineering specifications. Heretofore, specifications for structures included within the scope of the paper have been, to a great extent, the output of designers employed by manufacturers, and have shown the effects of the latter's bias.

If this discussion seems to be largely in the line of pointing out shortcomings in the author's specifications, it is not done in any spirit of criticism, but with a desire to see the specifications properly amplified and completed before they pass into a form which may be adopted as a standard. It is the writer's view that a technical specification should be something more than a mere check on the contractor; it should be a manual of design, complete enough to be a guide for the designer, whether he be employed by the purchaser or manufacturer, and should be so explicit and unambiguous as to permit of little discretion on the part of the designer, to the end that two independent designers working from it will produce substantially the same results.

The specifications seem to start with the assumption that the general dimensions of the structure are fixed. This should not be. For any given capacity and pressure there are certain economical ratios between height and diameter for elevated tanks, and there are other ratios for stand-pipes. Something relating to these rules of proportion should be included.

The specifications cover only stand-pipes and elevated tanks with hemispherical or flat bottoms supported on steel towers. One important form of elevated tank is ignored, that is, the tank with a semi-elliptical bottom and a small stand-pipe extending from the bottom to the ground, which serves both as a partial support for the tank and as a frost-proof inlet. This form is used largely for railroad water-station tanks, and the features which are peculiar to it should be included in this manual of design.

It is convenient to assume the weight of water at 62.5 lb. per cu. ft., because this makes convenient figures for calculation, namely, $8\frac{1}{3}$ lb. per gal. and $7\frac{1}{2}$ gal. per cu. ft. The use of two-thirds of the exposed area for computing wind pressure on cylindrical tanks is better practice than the use of one-half, as is done in many specifications. This ratio should be extended also to conical surfaces.

Table 1 allows a tension of only 12 000 lb. per sq. in. in tank plates, while a tension of 16 000 lb. per sq. in. is allowed in other parts of the structure. This is in accordance with common practice, and gives good results, although there is no apparent justification for the reduced unit stress. The tank plates are just as strong as structural shapes, and the loads coming upon them are just as capable of exact calculation. Of course, the ordinary reason for using a reduced unit stress is to provide extra section to cover corrosion and other forms

of deterioration. It would seem more rational to provide for this by an arbitrary addition to the thickness computed from the static pressure and net section. In fact, the writer has found it desirable and not any great hardship to specify an additional thickness of $\frac{1}{16}$ in. for tanks designed for a unit stress of 12 000 lb. Mr. Selby.

The allowable shear on rivets is given as 12 000 lb., with no stipulation that there shall be any reduction of this unit stress for rivets in tank plates. If the tank plates are designed for 12 000 lb. per sq. in. in tension, the rivets certainly ought to be designed for proportionately low unit stresses in shear and bearing. The allowed shear on field rivets and bolts is lower than that usually permitted. It is customary to prescribe that the number of field rivets shall be increased to 25% more than the number required for shop-driven rivets. On this basis the shear in field rivets would be 9 600 lb. per sq. in. instead of 9 000 lb.

No information is given in the specifications as to the efficiency of riveted joints. This is an important feature in designing tanks and boilers, as the computations, usually, are made directly on the basis of the efficiency of the joint, rather than by the more indirect process of computing the net section. It would be desirable to include a table showing the computed efficiencies of single- and double-lapped joints and single-, double-, and triple-riveted butt joints for various thicknesses of plates and diameters of rivets. Such tables are used by every manufacturer, and are published by some of them.

The use of the percentage of efficiency in the joint results in a simple formula for determining the thickness of plates in tanks subjected to hydrostatic pressure. Thus, if D = the diameter of the tank, in feet, H = the height, in feet, from the point considered to the surface of the water; the pressure, in pounds per square inch $= 0.434\ H$. The tension in the plate for 1 in. in height $= t = 2.6\ D\ H$. The required net area of the plate for 1 in. in height $= A = t \div f$, f being the unit stress, in pounds per square inch of net section. For $f = 13\,000$, $A = 2.6\ D\ H \div 13\,000 = 0.0002\ D\ H$. For an efficiency of two-thirds in the riveted joint, the gross area in 1 in. of height = the thickness of the plate, in inches $= A \div \frac{2}{3} = 0.0003\ D\ H$. To this should be added the additional thickness of $\frac{1}{16}$ in. already referred to. This formula, thickness of plate $= 0.0003\ D\ H$, makes a convenient and rapid method of determining the thickness of tank plates. For a unit stress in tension of 12 000 lb. the same formula results, if an efficiency of 72% is assumed.

It is customary to design on a basis of efficiency of at least 66%, and that would rule out all single-riveted lap joints. Section 14 specifies lap joints for vertical seams in plates $\frac{3}{8}$ in. thick and less, but does not specify whether they shall be single or double. They should be specified as double-riveted, with the possible exception of $\frac{1}{4}$-in. plates, in which the full strength of the plate is not developed.

Mr Selby. Double-riveted butt joints with the rivet diameters specified in Section 15, will secure an efficiency of at least 70% up to and including a thickness of $\frac{3}{4}$ in. For that reason it would seem desirable to design for an efficiency of 70%, and specify triple-riveted butt joints for plates having a thickness of $\frac{13}{16}$ in. or more.

The unit stress in compression should be limited to a maximum of 15 000 lb. for short columns.

Section 10 is ambiguous in that it allows unit stresses of 20 000 lb. in bracing, without specifying whether the unit stress referred to is for tension, compression, shear, or bearing. It gives a desirable addition to the bracing—which otherwise may be designed too light—to assume an arbitrary initial stress in all bracing members of 5 000 or 10 000 lb.

Section 12, specifying that plates forming the sides of cylindrical tanks shall be of different diameters, is not explicit. Of course, it means that the inside diameter of one plate shall be the same as the outside diameter of the next one.

In Section 13 it is not clear just what is meant by radial seams in a spherical bottom.

Section 16 specifies that plates more than $\frac{5}{8}$ in. thick shall be sub-punched and reamed, but does not specify the diameter of the punched hole and the amount to be removed by reaming.

Section 29 gives a rule for proportioning the stiffening member around the top of the tank, and specifies that the section modulus may be reduced under certain conditions, but does not specify definitely to what extent it may be reduced.

Section 35 allows an additional height of 6 in. for over-run. This is not clear. Presumably the meaning is that 6 in. in height shall be added to that required for the specified capacity in gallons.

Section 36 specifies that the bracing in towers shall be adjustable. Certainly riveted bracing for towers is better practice, although adjustable bracing may be allowable, on the score of economy, and the riveted bracing should not be excluded.

Section 37 specifies unit stresses in the anchor-bolts not exceeding 15 000 lb. per sq. in. Presumably, this refers to unit tensile strength, but it is not explicit. There is no specification covering the excess of the area at the root of the thread, and no specification which would produce good details in the design of the anchorage connections. Too many anchor-bolts are connected simply through the base plate of the column without provision for developing properly the full strength of the bolt, and this should be covered by a specification which will insure a good detail.

Section 38 specifies concrete sufficient in quantity to take the uplift, but does not provide any factor of safety. It is good practice to provide at least $1\frac{1}{2}$ or 2 times the computed uplift in the weight of the concrete. Furthermore, the specification should include a provision

that the arrangement of the anchorage in the concrete shall be such as to develop the full weight without depending on the tenacity of the concrete to hold the mass together; otherwise, this provision is apt to be overlooked, thus resulting in bad practice. Mr. Selby.

Sections 40 and 41 specify that the quality of materials and workmanship shall be according to certain specifications referred to by name. It would add little to the bulk of these specifications and much to their completeness if the specifications for quality of materials and workmanship were written out in full.

Section 42 specifies a shop coat of approved paint. It may be hopeless to try to get the Engineering Profession to agree on an approved paint, but probably red lead would more nearly satisfy the ideas of most members than any other one kind.

The specification has covered only the work of the designer and manufacturer of tanks, but, inasmuch as most contracts for tanks and stand-pipes, especially for municipalities, include their erection, it would seem desirable to give specifications for erection, testing, and field painting.

It is to be regretted that the author did not see fit to give some text with the specifications.

CHARLES C. HOPKINS, M. AM. SOC. C. E. (by letter).—The author expresses the wish that the general specifications, as submitted in his paper or as finally revised, be adopted by the Engineering Profession. It appears to the writer that, in their present form, the specifications are not drawn as carefully as they should be; that some portions might be omitted, others added, or added to, and other advantageous changes made. Mr. Hopkins.

Although the author evidently intends that there shall be a distinction between a stand-pipe and a tank, he has used the word, "tank," in places where either one is or should be meant, and, in other places, where only an elevated tank can be intended.

It is inferred that by "tower" is meant the framework supporting an elevated tank, that a "tank" may be thus elevated or may stand with its bottom at or near the surface of the ground, and that a "stand-pipe" is simply a tank without a supporting tower, but having a height considerably greater than its diameter.

As the author uses the word "tank" at times in a general sense, including stand-pipe, it leaves considerable uncertainty in other places, for example, in Section 31. If it is assumed that stand-pipes are to be roofed, issue might be taken with the author that they are not generally roofed in the colder climates unless enclosed in a building or "tower," because the floating of the ice which forms around the inside would probably lift, and thus ruin, the roof.

The writer suggests that the weight of water should not be assumed to be 63 lb. per cu. ft., when 62.5 lb. is much more nearly correct and

Mr. Hopkins. a more convenient number to use; or, what would be better still, that Section 3 be stricken out entirely as being unnecessary.

Section 12 appears to be drawn rather carelessly, in several respects. Why refer to the tanks as "cylindrical"? Does not this section refer to stand-pipes also? What is meant by the plates being of different diameters? Possibly the intention is that the courses forming the sides shall lap over each other inside and outside alternately. What is meant by the expression, "No foreign material shall be allowed when caulking"? In specifying that in oil-tank work "both the inside and outside of the tanks shall be beveled for caulking," the author probably means that the inside and outside edges of the plates shall be beveled for caulking; if so, the wording of the section should be changed.

Although the lapping of the side plates alternately inside and outside is the usual construction, what material objection is there to making each horizonal course (especially in small stand-pipes) slightly conical, so that the top of one course enters the bottom of the one next above it? The writer asks this for information, as he has heard a manufacturer of stand-pipes state that he preferred to build in that way, believing it would be more satisfactory, as all the inside caulking of the horizontal joints would be over-handed, and would be done more easily and, perhaps, better.

The author's specifications do not state the manner in which the anchor-bolts are to be fastened to the tank, or the manner in which the bottom of the stand-pipe is to be fastened to the side plates, possibly leaving these points, with others, to be shown on a plan to be submitted, although there is no reference to such a plan in the specifications. As to the connection of the bottom with the side, it is the writer's opinion that for large stand-pipes the use of both inside and outside angles is best, riveting the bottom plates as well as the side plates, to both angles. If either angle is omitted, as it may well be at times, it should be the inner one, because an outside bottom angle gives the stand-pipe a slight additional factor of safety against overturning, and a leak occurring at the angle can be caulked without drawing off the water. Thus far, the writer has always found the outside angle entirely satisfactory.

It is suggested that the specifications should state that all caulking edges shall be planed to a proper bevel, and that all caulking shall be done with a round-nosed tool.

The specifications for the manhole, roof, trap-door, and other details are rather meager, and might perhaps be left out altogether or only made the basis of plans to be submitted by the contractor and to be approved afterward by the engineer. The latter method is not usually as satisfactory as to submit definite plans and specifications on which all contractors may bid.

Mr. Hopkins.

The meaning of Section 35 is not clear to the writer. Certainly several different meanings could be ascribed to it. If it is assumed that the engineer for the "owner" is designing the foundations on which the stand-pipe is to rest, then Section 38 might well be omitted; but if the contractor for the stand-pipe is to build the foundations also, then a complete specification for concrete should be added to Section 38. Unless the contractor for the stand-pipe, tank, or tower, is to build the foundation, Section 11 and Table 2 might as well be omitted, especially the first half of Table 2, because the engineer in charge must satisfy himself as to the safe loads, and for these no general specifications, such as those proposed by the author, should govern.

In reference to painting, the writer believes that such inaccessible parts as "laps in contact" should be either painted or oiled before leaving the shop. Specifications prepared by him and his associates have usually provided that, before leaving the shop, all steel shall be thoroughly cleaned of rust and scale, and be thoroughly coated with boiled linseed oil; that inaccessible surfaces shall be painted before erection; that, after the stand-pipe has been tested and found to be tight, it shall be painted inside and outside with the paint (or varnish) specified; and that no painting shall be done in wet or freezing weather.

Section 16 states that "plates more than $\frac{5}{8}$ in. thick shall be sub-punched and reamed." The writer believes that all punching of plates should be done from the surface which is to be in contact, and that in plates as thick as $\frac{3}{4}$ in., or thereabouts, the rivet holes should be drilled.

The Manufacturers' Standard Specifications, as revised on February 6th, 1903, are excellent for workmanship, as far as they go, but for stand-pipe and tank work they are not as complete as they should be. The writer would suggest the following, as some of the requirements in workmanship, which are more or less necessary, and are not covered by the Manufacturers' or the author's specifications.

All plates and angles shall be shaped to the proper curve by cold rolling; heating or hammering for straightening or curving shall not be allowed, and no scarfing or other work shall be done on the steel below a specified temperature. The diameter of the die used in punching rivet holes shall not exceed that of the punch by more than $\frac{1}{16}$ in. All rivet holes in horizontal seams of stand-pipes may be punched. All parts shall be adjusted to a perfect fit and shall be marked before leaving the shop. Eccentricity of rivet holes, if not sufficient to cause rejection, shall be corrected by reaming, and rivets of larger size shall be used in the holes thus reamed. The engineer or inspector shall have the right to require driven rivets to be cut out at random in order to inspect the quality of the workmanship. Rivet caulking shall not be allowed, but leaky rivets shall be cut out and replaced with new ones.

Mr. Hopkins. The riveting shall not crack the plates or angles. The plates forming the courses shall be as long as practicable.

Inasmuch as the Manufacturers' Standard Specifications for workmanship are short, it might be well to print them entire, or perhaps only the necessary parts, with any others which might be thought advisable, thus having convenient, for the inspector on erection, the complete specifications governing the work under his charge.

The author's specifications omit mention of the inlet (or outlet) and overflow pipes of stand-pipes, and, although they describe a ladder for a tank, they do not mention one for a stand-pipe—quite a necessary adjunct. It is evident that there should not be an inside ladder in an unenclosed stand-pipe where ice is likely to form.

Assuming that general specifications are needed—of which there may be some doubt—it seems to the writer that, to avoid confusion, it might be best to separate the proposed specifications into specifications for steel stand-pipes and specifications for elevated steel tanks, or to place them under separate headings in the same specifications, following some general specifications of such parts as are common to both.

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AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

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A SYSTEM OF COST KEEPING.

Discussion.*

By Messrs. P. L. Reed, W. C. Hammatt, Emile Low, Richard T. Dana, and T. Kennard Thomson.

P. L. Reed, M. Am. Soc. C. E. (by letter).—As the author points out, cost records are of greatest value to those who make them and at the time and place at which they are made. Their value diminishes rapidly with time, distance, and changing personnel, for, while it may be permanently recorded that a piece of work cost a definite sum, it becomes increasingly difficult to determine just what that piece of work was, and all that was involved in its construction. The numerator is known, but the denominator is often most uncertain, and the quotient is what is needed as a guide. Records involve the expense of preparation and the mental effort of examination and future reference, and, as costs are amplified, the expense gains on the utility, sometimes to the point where general conclusions, which might be of considerable value, are drowned in masses of details of slight transient importance, which rapidly lose their value until, in practice, the records are not found worthy of the time and mental effort required to consult them. Mr. Reed.

The deviser of a cost-keeping system, which, in some form, is almost essential to the conduct of any business where things are made, must curb his enthusiasm by considerations such as these, and stop before he has gone too far.

*This discussion (of the paper by Myron S. Falk, M. Am. Soc. C. E., printed in *Proceedings* for March, 1909, is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

Mr. Reed. Referring to some of the details of the system described, which seems to be a very good one, the writer occasionally sees a voucher similar to Figs. 1 and 2, on the outside of one fold of which is printed the check upon which the bank makes payment. This seems to have some advantages, as the endorsement on the back of the folded voucher is likewise a receipt for the amount of the bill as summarized on one fold and distributed on the other, much as in the case of that illustrated by the author. The form would also be most convenient for auditing, though, possibly, not so popular with the banks.

The preparation of the time report, Fig. 4, on a fresh card or cards each day, with names, rates of pay, etc., would seem to involve an amount of clerical work out of proportion to its value. If entries are made day by day, the weekly pay-roll form shown in Fig. 9 apparently serves every purpose of that shown in Fig. 4, except to inform the head office daily of the name, hours worked, and rate of pay of each employee, which does not seem very important where the condensed statement of the force account by trades is sent in daily on the form, Fig. 5, and the names, etc., weekly, on the form, Fig. 9.

Where the force is large, or where the work of time and cost keeping is divided between timekeepers and cost clerks, it is necessary to assign numbers or other simple characters to the different sub-heads under which the work is divided, and to enter these characters opposite the names of employees on the time card, so that the labor charges can be properly distributed by one not in close touch with the work on the ground.

Mr. Hammatt. W. C. HAMMATT, ASSOC. M. AM. SOC. C. E. (by letter).—When in charge of various pieces of construction, the writer has always endeavored to keep close and accurate costs, in order to check, not only the unit costs of different classes by previous work of the same class, but also to present to the company, at the end of the work, an accurate and minutely itemized valuation of the work done, for inventory and insurance purposes. He has used practically the same system in his work as Superintendent of Construction for the Mountain Copper Company, at Martinez, the American Smelting and Refining Company, at Chihuahua, Mexico, and the Mammoth Copper Mining Company, at Kennett, Cal.; therefore, only one of these cases, for example, the construction at Chihuahua, will be described.

The first step of the system involved giving a segregation number to each piece of work. This minimized the clerical work, and also gave convenient reference to ledger accounts. In this system of segregation whole numbers are used for the main divisions of work and decimals of various length for the sub-segregations under these divisions. This provided for a combination of all the decimals of a certain degree when segregations of lesser refinement were necessary for certain purposes. For example:

CONSTRUCTION SCHEDULE. Mr. Hammatt.

General Expense:

01 Superintendence.
02 Surveying and drafting.
03 Office expenses.
04 Proportion, main office.
05 Temporary tracks.
06 Tools, push cars, concrete mixer.
07 Real estate.
08 Warehouse stock.
09 Warehouse expense.

Blast Furnaces:

10 Building.
11 Furnace No. 4, bustle pipe, settler, etc.
12 Changes in present furnaces.
13 Slag and matte equipment.
14 Furnace No. 5, bustle pipe, settler, etc.
15 .....
16 .....
17 .....
18 Connecting blast pipe.
19 Blast pipe in smelter.

(These sub-headings apply to all following headings.)

0.1 Grading.
0.2 Forms.
0.3 Concrete.
0.4 Superstructure.
0.5 Falsework.
0.6 Plumbing.
0.7 Wiring.
0.8 Painting.

Converter Plant:

20 Building (including crane).
21 Converter No. 1 (including stand and tilting system).
22 " No. 2 (" " " " ").
23 Downtakes.
24 Flue.
25 Charge system.
26 Copper disposal.
27 Slag disposal.
28 Blast pipe.
29 Hydraulic system.

Relining System:

30 Building.
31 Relining pit (including tamper and crane).
32 Clay mills.
33 Clay and quartz bins.
34 Track and trestle to same.
35 Tunnel.
36 Drying stands.
37 .....
38 .....
39 .....

Mr. Reed. Referring to some of the details of the system described, which seems to be a very good one, the writer occasionally sees a voucher similar to Figs. 1 and 2, on the outside of one fold of which is printed the check upon which the bank makes payment. This seems to have some advantages, as the endorsement on the back of the folded voucher is likewise a receipt for the amount of the bill as summarized on one fold and distributed on the other, much as in the case of that illustrated by the author. The form would also be most convenient for auditing, though, possibly, not so popular with the banks.

The preparation of the time report, Fig. 4, on a fresh card or cards each day, with names, rates of pay, etc., would seem to involve an amount of clerical work out of proportion to its value. If entries are made day by day, the weekly pay-roll form shown in Fig. 9 apparently serves every purpose of that shown in Fig. 4, except to inform the head office daily of the name, hours worked, and rate of pay of each employee, which does not seem very important where the condensed statement of the force account by trades is sent in daily on the form, Fig. 5, and the names, etc., weekly, on the form, Fig. 9.

Where the force is large, or where the work of time and cost keeping is divided between timekeepers and cost clerks, it is necessary to assign numbers or other simple characters to the different sub-heads under which the work is divided, and to enter these characters opposite the names of employees on the time card, so that the labor charges can be properly distributed by one not in close touch with the work on the ground.

Mr. Hammatt. W. C. Hammatt, Assoc. M. Am. Soc. C. E. (by letter).—When in charge of various pieces of construction, the writer has always endeavored to keep close and accurate costs, in order to check, not only the unit costs of different classes by previous work of the same class, but also to present to the company, at the end of the work, an accurate and minutely itemized valuation of the work done, for inventory and insurance purposes. He has used practically the same system in his work as Superintendent of Construction for the Mountain Copper Company, at Martinez, the American Smelting and Refining Company, at Chihuahua, Mexico, and the Mammoth Copper Mining Company, at Kennett, Cal.; therefore, only one of these cases, for example, the construction at Chihuahua, will be described.

The first step of the system involved giving a segregation number to each piece of work. This minimized the clerical work, and also gave convenient reference to ledger accounts. In this system of segregation whole numbers are used for the main divisions of work and decimals of various length for the sub-segregations under these divisions. This provided for a combination of all the decimals of a certain degree when segregations of lesser refinement were necessary for certain purposes. For example:

CONSTRUCTION SCHEDULE. Mr. Hammatt.

General Expense:

01 Superintendence.
02 Surveying and drafting.
03 Office expenses.
04 Proportion, main office.
05 Temporary tracks.
06 Tools, push cars, concrete mixer.
07 Real estate.
08 Warehouse stock.
09 Warehouse expense.

Blast Furnaces:

10 Building.
11 Furnace No. 4, bustle pipe, settler, etc.
12 Changes in present furnaces.
13 Slag and matte equipment.
14 Furnace No. 5, bustle pipe, settler, etc.
15 .....
16 .....
17 .....
18 Connecting blast pipe.
19 Blast pipe in smelter.

(These sub-headings apply to all following headings.)

0.1 Grading.
0.2 Forms.
0.3 Concrete.
0.4 Superstructure.
0.5 Falsework.
0.6 Plumbing.
0.7 Wiring.
0.8 Painting.

Converter Plant:

20 Building (including crane).
21 Converter No. 1 (including stand and tilting system).
22 " No. 2 (" " " " ").
23 Downtakes.
24 Flue.
25 Charge system.
26 Copper disposal.
27 Slag disposal.
28 Blast pipe.
29 Hydraulic system.

Relining System:

30 Building.
31 Relining pit (including tamper and crane).
32 Clay mills.
33 Clay and quartz bins.
34 Track and trestle to same.
35 Tunnel.
36 Drying stands.
37 .....
38 .....
39 .....

Mr. Hammatt. *Charging System:*

40 Tunnel.
41 Trestle.
42 Trackwork.
43 Equipment, charge cars.
44 .....
45 .....
46 .....
47 .....
48 .....
49 .....

Flue System:

50 Dust chamber, steelwork (including hoppers).
51 .....
52 .....
53 .....
54 Tracks.
55 New stack.
56 Equipment, cars, etc.
57 .....
58 .....
59 .....

Power Plant and Blast Mains:

60 North addition (building).
61 South " (").
62 Receiver.
63 Changes in present blowers and connections.
64 Blower No. 4 and connections and motor.
65 " No. 5 " " " "
66 Blowing engine.
67 Blower No. 6, connection and motor.
68 " No. 7, " " "
69 Generating set No. 2.
70 Switch-board.
71 .....
72 .....
73 .....
74 .....
75 .....

Sampling System:

76 Temporary sampling room.
77 .....
78 .....
79 .....

Mr. Hammatt.

Track System:

80 Standard gauge changes.
81 New standard gauge yard.
82 Slag tracks.
83 Lime track.
84 Track scales.
85 .....
86 .....
87 .....
88 .....
89 .....

0.1 Grading.
0.2 Track-laying.
0.3 Ballasting.
0.4 Switching.

Shops:

90 Machine-shop building and blacksmith shop.
91 " " plant.
92 Steel " building.
93 Compressor-motor and building.
94 Boiler-shop and round-house building.
95 " " " " " plant.
96 Carpenter shop building.
97 " " plant.
98 Electrical shop building.
99 .....

Bins and Trestles:

100 Main receiving bins.
101 Trestle approach to same.
102 .....
103 .....
104 .....
105 .....
106 .....
107 .....
108 .....
109 .....

Miscellaneous:

110 High-pressure pipe line.
111 Cement sheds.
112 Greek bunk house.
113 Drain tunnel.
114 Smelter stable.
115 .....
116 .....
117 .....
118 .....
119 .....

Mr. Hammatt. *Standard Gauge Equipment:*

120 Oil tank.
121 Locomotives.
122 Wrecking crane.
123 Cars.
124 .....
125 .....
126 .....

In regard to this list, it may be stated that it is faulty in its arrangement, due to the full plans for construction not being complete at the time the work was started. The writer believes in a more complete segregation of fixed and movable plant, for insurance and appraisal purposes, as a machine can be moved to a new site, while its foundation can not, and the superstructure of a building is subject to destruction and damage by fire, while its foundation is not.

AMERICAN SMELTING & REFINING CO.
CHIHUAHUA PLANT

Daily report of labor on_____
for_____
Segregation_____
Date_____

NO.	EMPLOYEES	NO. HRS.	RATE	AMOUNT
	Foreman			
	Carpenters			
	"			
	Helpers			
	Mechanics			
	"			
	"			
	Helpers			
	Stone Masons			
	Brick "			
	Helpers			
	Engineers			
	Firemen			
	Plumbers			
	Helpers			
	Electricians			
	Laborers			
	"			
	Water boys			

FOREMAN

Standard size, 7 by 3¼ in.

FIG. 10.

AMERICAN SMELTING & REFINING CO.
CHIHUAHUA PLANT

Cost report for_____
_____on_____
Segregation_____
Ending_____

DATE	ACCOUNT	UNITS	RATE	AMOUNT
	Labor			
	do			
	do			
	do			
	do			
	do			
	do			
	Brick			
	Stone			
	Sand			
	Lime			
	Cement			
	Lumber			
	Cost per unit			

Standard size, 7 by 3¼ in.

FIG. 11.

In the matter of construction reports: The forms of two reports used for cost keeping from day to day, are given in Figs. 10 and 11.

Mr. Hammatt.

That shown by Fig. 10 was made out daily by the foreman for each kind of work done under his supervision, a separate report being made out for each segregation. At the end of the week the form shown by Fig. 11 was made out by the cost clerk from the data on Fig. 10, and the measurement or estimate of each piece of work performed. The form shown by Fig. 10 was also used by the timekeeper as a check on his time, as the total number of man-hours or man-days should agree in both cases, and also the total in dollars and cents for the pay period.

As there is likely to be inaccuracy in progress measurements which are taken daily, or even weekly, it is well to combine the reports of several weeks. These inaccuracies will then disappear, and a truer calculation of unit cost may be made.

This system had to be varied for the different conditions under which the construction work was conducted, for example, in Mexico, where the foremen were mostly illiterate, special men were employed for the cost keeping, and the forms differed slightly from those used in places and on classes of work where men of better mental capacity were employed.

Bills for materials of construction were distributed by the superintendent or cost clerk at the time of checking to the accounts where they belonged. When bills could not be charged to their regular jobs, on account of the actual quantity of material going to each job being unknown, the total was charged to warehouse, and the different jobs debited and warehouse credited as the material was issued.

It may be added that the engineer generally works under difficulties in keeping accurate cost records, as the general idea among American employers is that such data are useless expense, and they will not allow the necessary clerical force to make them accurate and useful. The sentiment among most of them was expressed in the late Mr. T. S. Austin's remark to the writer that whether the brickwork costs $12 or $30 per 1 000, the work would go on until it was completed or the company "went broke."

Mr. Low.

Emile Low, M. Am. Soc. C. E. (by letter).—The system of cost keeping described in Mr. Falk's paper is that used by a contracting company, and is really a compilation of expenses. It shows how the money was expended among different assumed items, and thus affords data for the detection of mismanagement in work of similar nature.

On construction work it is always interesting for the engineer-in-charge to know, at least approximately, what the work is costing the contractor. For want of absolute knowledge of the cost of some items, it may be difficult to ascertain the total cost, but even a knowledge of the cost of some of the items, as before stated, is of some satisfaction.

It may be stated with some assurance that, during construction,

Mr. Low. nearly every railroad company keeps some kind of a force account, which is rendered to the chief engineer, generally weekly, by his subordinates.

Some years ago, while the writer was in charge of some heavy railroad construction in Southwestern Virginia, he originated and put into effect a system of cost keeping, which showed monthly the cost to the contractor of various classes of work, the statements of which were submitted with the monthly estimates.

The system was extremely simple, and consisted essentially of three blank forms which constituted the basis for keeping the records.

On the railroad in question, each residency consisted of from seven to ten 1-mile sections, thus each residency ranged from 7 to 10 miles in length. The contractor's forces were counted daily by a timekeeper employed by the railroad company, which work comprised his whole duty.

A special timebook was prepared, a double page of which is shown by Fig. 12.

From an inspection of the timebook it will be noted that it is arranged to keep a separate record of each and every piece of work, no matter what it may be. It is also aranged to show the totals for each full week as well as for the partial week. Occasionally, there is a month which includes four full weeks and parts of two weeks, making space for six entries necessary.

The usual practice is to send in the force accounts weekly to headquarters, and the arrangement of the book facilitates this. At the end of the month there is a summary from which the cost of the work is obtained.

Of course, owing to the method used, the cost obtained is not absolute, as the force employed during the morning and afternoon may vary in numbers, the timekeeper passing over the work only once a day. Where absolute accuracy is required, more timekeepers would have to be employed.

As there are no printed headings, the spaces may be filled in as the nature of the work requires. Space is provided for ten different designations of labor and material, and in cases where there are more items, other pages can be used. It will be seen that the timebook is very flexible, and can be adapted to any contingency.

At the end of the month all work is classified, the form, Fig. 13, being used. The sample sheet shows columns for clearing, grubbing, grading, and masonry. This column is left blank, and the various classes of work are written in, using as many sheets as necessary.

Before the final classification, by sections, the location of each individual piece of work is entered in the column having the printed heading, "Sec." On the grading there may be as many as ten or more forces at work on a mile section, several culverts, bridge abutments, and piers may also be building.

NORFOLK & WESTERN R.R. CO. _____ DIVISION

DAILY STATEMENT OF FORCE EMPLOYED BY

_____ CONTRACTORS

ON _____ STATION _____

SECTION NO. _____

FOREMAN

DATE		DESIGNATION OF LABOR AND MATERIAL: 1	2	3	4	5	6	7	8	9	10
	F										
	S										
	TOTAL										
	S										
	M										
	T										
	W										
	T										
	F										
	S										
	TOTAL										
	S										
	M										
	T										
	W										
	T										
	F										
	S										
	TOTAL										

DATE		DESIGNATION OF LABOR AND MATERIALS: 1	2	3	4	5	6	7	8	9	10
	S										
	M										
	T										
	W										
	T										
	F										
	S										
	TOTAL										
	S										
	M										
	T										
	W										
	T										
	F										
	S										
	TOTAL										

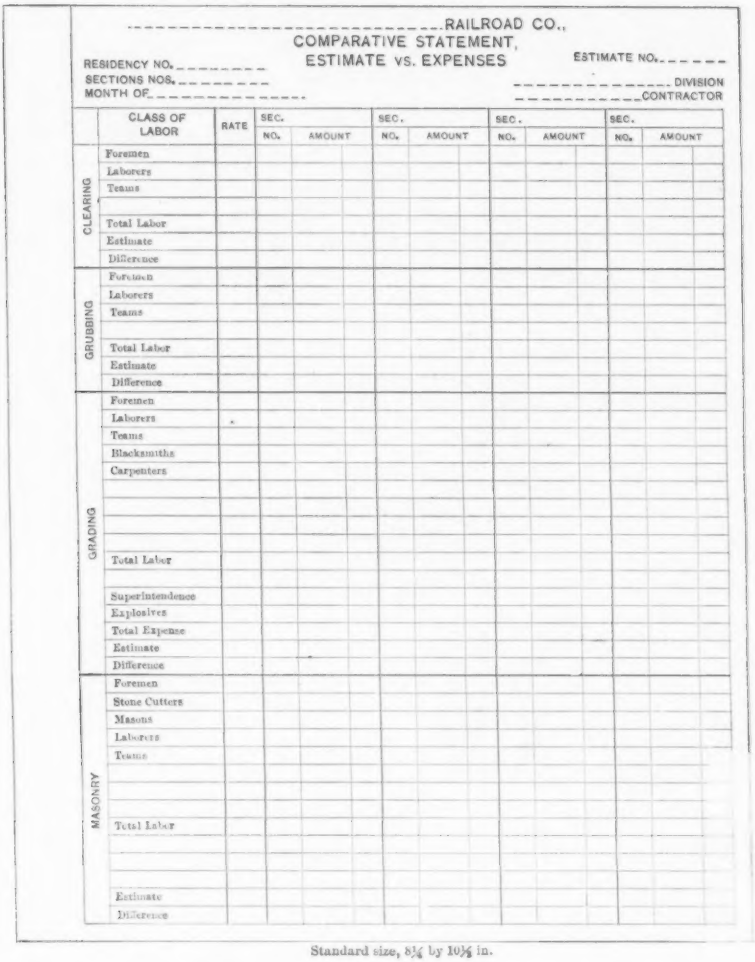
SUMMARY

DESIGNATION OF LABOR AND MATERIAL	FOR WEEK AND PARTIAL WEEK ENDING:					TOTAL	RATE	AMOUNT	
1									
2									
3									
4									
5									
6									
7									
8									
9									
10									
TOTAL									

Standard size, 7 by 8½ in.

FIG. 12.

Mr. Low. The blank form, Fig. 13, may be used in making a compilation showing the cost for separate localities, or the totals for any one section may be entered, these totals being obtained by the addition of the various separate items.



_____.RAILROAD CO.,
COMPARATIVE STATEMENT,
ESTIMATE VS. EXPENSES

RESIDENCY NO._____ ESTIMATE NO._____
SECTIONS NOS._____ _____ DIVISION
MONTH OF._____ _____CONTRACTOR

	CLASS OF LABOR	RATE	SEC. NO.	AMOUNT	SEC. NO.	AMOUNT	SEC. NO.	AMOUNT	SEC. NO.	AMOUNT
CLEARING	Foremen									
	Laborers									
	Teams									
	Total Labor									
	Estimate									
	Difference									
GRUBBING	Foremen									
	Laborers									
	Teams									
	Total Labor									
	Estimate									
	Difference									
GRADING	Foremen									
	Laborers									
	Teams									
	Blacksmiths									
	Carpenters									
	Total Labor									
	Superintendence									
	Explosives									
	Total Expense									
	Estimate									
	Difference									
MASONRY	Foremen									
	Stone Cutters									
	Masons									
	Laborers									
	Teams									
	Total Labor									
	Estimate									
	Difference									

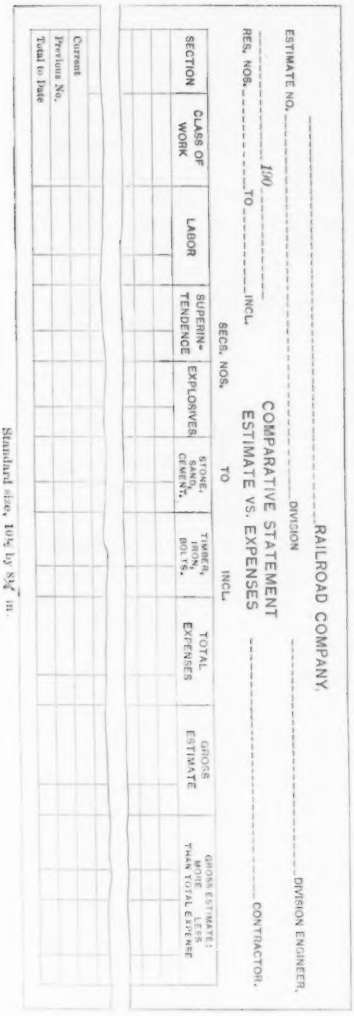
Standard size, 8¼ by 10½ in.

FIG. 13.

Fig. 14 is a blank form showing the summary of all the work on a contract.

The main idea underlying the whole scheme is to make a monthly comparison with the engineer's estimate.

Mr. Low.



RAILROAD COMPANY.

ESTIMATE NO. _____ DIVISION _____ DIVISION ENGINEER.

_____ 190 _____

RES. NOS. _____ TO _____ INCL. COMPARATIVE STATEMENT _____ CONTRACTOR.

ESTIMATE VS. EXPENSES

SECS. NOS. _____ TO _____ INCL.

SECTION	CLASS OF WORK	LABOR	SUPERINTENDENCE	EXPLOSIVES	STONE, SAND, CEMENT	TIMBER, IRON, BOLTS	TOTAL EXPENSES	GROSS ESTIMATE	GROSS ESTIMATE LESS THAN TOTAL EXPENSE
Current									
Previous No.									
Total to Date									

Standard size, 10¾ by 8¼ in.

FIG. 14.

FORM No. C.E. _____

CONSTRUCTION DEPARTMENT.

RAILROAD COMPANY.

_____ DIVISION.

COMPARATIVE STATEMENT.

ESTIMATE VS. EXPENSES.

RES. NOS. _____ TO _____ INCL. _____

SECS. NOS. _____ TO _____ INCL.

ESTIMATE NO. _____

_____ 190 _____

SUMMARY: CONTRACTOR

	DOLLARS	CTS.
Labor		
Superintendence		
Explosives		
Stone, Sand, Cement		
Timber, Iron, Bolts		
Total Expenses		
Gross Estimate		
Apparent *		

*GAIN OR LOSS

Total Expense		
Net Estimate		
Apparent		

EXTRA WORK

Amount plus 15 Per Cent		
Force Account		
15 Per Cent		

Certified By

_____ DIVISION ENGINEER

Standard size, 3¼ by 8¼ in. Back of Form shown by Fig. 14

FIG. 15.

Mr. Low. Referring to the timebook, Fig. 12, there is on a page the cost to the contractor of a particular piece of work, during the month, say, a bridge abutment. An estimate of quantities and cost has been made by the engineer in charge of this particular work.

The details of the cost to the contractor are transferred to the blank form, Fig. 13, the amounts being shown under the heading marked "Sec.," the "estimate" placed under the total, and the difference obtained, which will be more or less as the case may be. These statements then show in detail the expense and estimate of each particular piece of work.

Then a summary is made of each and every kind of work by sections (1 mile), and also a summary of the various estimates. These summaries then show, mile by mile, the status of the various kinds of work.

The engineer's estimate always shows the total cost of a section, and in order to make a proper comparison of all expense, another blank form is needed, which shows a summary of all the labor, superintendence, explosives, stone, sand, cement, timber, iron, bolts, etc., this being the total cost of all the work in a section.

These totals, shown by Fig. 15, may be readily obtained by using the form shown by Fig. 14, for the purpose of tabulation, as the main items are all placed in separate columns, and can be easily added. In some quarters objection is made to keeping an accurate account of the forces employed by the contractor and the tabulated cost, the main reason assigned being that the engineer's estimate is influenced thereby to some extent. This need not be, as the cost to the contractor is not generally known until some time after the estimate is made out. In most public works nowadays there is no classification of excavation, so there are no grounds whatever for manipulation, although there might be under the classification method of dividing excavation into earth, hardpan, and loose and solid rock.

Again, the contractor's costs may be tabulated wholly in the division engineer's office, or even in the office of the chief engineer, he being furnished with duplicate copies of the timebooks, which, as before stated, form the basis of the whole system described.

The writer is a firm believer in keeping costs of contractor's operations, for the very important reason (and almost the only one), that it affords some tangible evidence of fixing suitable unit prices on future work, instead of using that very undesirable makeshift, guessing at them.

Mr. Dana. Richard T. Dana, M. Am. Soc C. E. (by letter).—The author states that "it seems hardly necessary to call attention to the very small value that cost records may have to others than to those who gathered them and who realized and understood fully every local condition at the point of construction." Apparently, he assumes that it is not possible

to record the local conditions with sufficient thoroughness to make such records of value to a man who has not seen the site of the work. Such an assumption involves the further proposition that it is possible to appreciate conditions which it is impossible to express or describe in writing. If the writer is correct in this interpretation of the author's meaning, the author's position is untenable. It is generally accepted that in shop work the conditions can be sufficiently well recorded as to have resulted in the reduction of the art of cutting metals, for example, to almost an exact science.* Mr. Dana.

In out-of-door work there is necessarily interference by the weather, by variation in the conditions of the materials to be handled, by accidents to large units of equipment, and by a considerable number of other causes which do not obtain in the shop. These causes, however, are such as to admit of classification, and it is entirely possible to obtain fairly approximate quantitative expressions for their comparison. The outside field has a great advantage over the shop in that the aid of photographs can be enlisted, and, at an almost infinitesimal cost, photographic records, which are more accurate than the eye and much more durable than the memory, can be obtained, and these records can help out the written notes so greatly as to enable the compilation of practically complete information for future use.

The author's assumption that a man must have seen work done in order to make cost records of it of value to him, means that his memory—his uncertain, hazy, and, perhaps, prejudiced memory—is to be trusted in preference to cold, hard, statistical facts. The writer's experience has been altogether in the opposite direction.

In the third paragraph, the author states that by the use of cost records in the field a contractor can with reasonable accuracy compare the cost of work performed in any specified period with that of any other period. As the local conditions necessarily vary from time to time, it seems difficult to appreciate why the cost from period to period can be compared with benefit, when, according to the author's statement in the preceding paragraph, cost records have small value to others than to those who gathered them, unless the contractor is going to keep the same superintendent on the job from start to finish.

The author has given some blanks showing the method of obtaining cost in the field. It appears as if he had endeavored to combine a cost-keeping system with a bookkeeping system. The bookkeeping system is based on the use of only one unit, the dollar, while cost keeping, as the term has been used in recent years, involves the recording of efficiency, and may utilize any of a considerable number of units. Thus, the cost of drilling may be expressed in minutes per foot; the cost of hauling material may be expressed as the reciprocal of the

* Note the papers by Mr. F. W. Taylor in *Transactions*, Am. Soc. Mech. Engrs., Nos. 1003 and 1119.

Mr. Dana. expression "tons per team-day," and so on. The writer prefers to consider cost keeping as simply a method for recording efficiencies, using whatever units seem most convenient at the time, and to confine it to a class by itself, not allowing it to interfere in any way with the bookkeeper's work, thus avoiding the imposition of a considerable amount of burdensome labor upon the bookkeeper, and using the units most suitable for the case in hand.

Because rates of labor vary in different localities at the same time and at different times in the same locality, the dollar is not even approximately a true measure of efficiency, and the writer prefers to use, as a standard for comparison, the United States Reclamation Service unit of a man-day.

Mr. Thomson. T. KENNARD THOMSON, M. AM. SOC. C. E.—The speaker is a firm believer in cost analyses, provided all the facts are known, including the personal characteristics of the contractor and his superintendent. For all know that two contractors can take exactly similar contracts under exactly similar conditions as regards labor, material, location, etc., and at the same prices; and while one contractor will make a good profit, the other will sustain a heavy loss, just as one man can start a grocery store or a restaurant, and make his fortune, while another man with the same capital, store, and stock will go bankrupt.

Again, the same contractor's profit or loss will depend largely on his superintendent—unless he acts as his own superintendent. As an example, a well-known contractor had a steam-shovel job with a $5 000 superintendent who left when the job was half through, and a $3 000 man was put in his place. The second man, with exactly the same labor, cars, tools, material, etc., only turned out 60% as much material as his predecessor.

Two of the speaker's clients are contractors—though not competitors—in similar lines of business, and he always advises one to bid 30% higher than the other, and, even then, he feels sure that the first will make a bigger profit than the second.

While cost analysis, therefore, is absolutely necessary, and is much used by the speaker, it would generally be disastrous for a contractor to bid on work in which he was inexperienced, on the strength of the cost analysis made by some one else, and, while the speaker is in favor of cost analysis, he generally wants to know who made it and who will use it.

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AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

THE MAXIMUM WEIGHTS OF SLOW FREIGHT TRAINS.

Discussion.*

By L. A. Riley, 2d, Esq.

L. A. Riley, 2d, Esq. (by letter).—A close analysis of Mr. Bissell's paper shows some disagreement with observed data, and the writer believes also with theory. The accuracy of Mr. Bissell's weights of trains on compensated grades is not questioned, but they appear to be somewhat low, considering the weight of the locomotive he uses for estimating purposes. Mr. Riley.

The writer's notebook is full of record hauls, but it is not thought that they would serve any useful purpose in this discussion. It might be interesting, however, to give (in Table 4) the tonnage rating of a consolidation-type, slow-freight locomotive, which rating has stood the test of more than a year's operation under all sorts of conditions.

The locomotive in question, as may be seen from Table 4, was considerably lighter than Mr. Bissell's.

That the majority of the figures in Table 4 were obtained in practice the writer can testify, and he can also testify to the fact that at such times no sand was used to increase the adhesion, which was calculated at 25% in making up the rating. The winter rating was for dry track, the only change in calculation being an increase of the rolling friction. In case of icy rails, or stormy weather, it was subject to a further reduction. All of which leads to a disagreement with the 20% adhesion and the theoretical deduction for the same, as given

*This discussion (of the paper by C. S. Bissell. M. Am. Soc. C. E., printed in *Proceedings* for February, 1909) is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

Mr. Riley. by Mr. Bissell. He states that "the tractive power was to be reduced by the proper amount for a constant speed of 10 miles per hour," and, later, that 13.7% was taken as the proper reduction in the tractive power for a speed of 10 miles per hour.

TABLE 4.—FIFTY-CLASS ENGINES; CONSOLIDATION TYPE; CYLINDERS, 22 BY 28 IN.; STEAM PRESSURE, 200 LB.; TOTAL WEIGHT, 144 TONS; WEIGHT ON DRIVERS, 155 156 LB.

Grade.	RATING:	
	Summer.	Winter.
0.38%	2 850	2 485
0.5%	2 424	2 155
0.7%	1 940	1 940
1.0%	1 492	1 386
1.1%	1 386	1 292
1.2%	1 292	1 212
1.4%	1 140	1 078

Compensated actual grades. Speed, 12 miles per hour. Rolling friction, 6 and 8 lb. per ton, respectively. Track in fair condition.

It appears that he calculates the rate of adhesion as 23.2% from the tonnage records, and then reduces this rate by 13.7% for a speed of 10 miles per hour. Sliding friction is subject to a reduction at increased speeds, but the writer does not know where Mr. Bissell finds authority for a reduction in the rate of adhesion with change in speed.

If the rate of adhesion is 23.2% at zero miles per hour, it remains practically the same at any speed. In other words, the tractive power of the locomotive will follow the adhesion from zero speed up to a critical speed at which the locomotive fails to supply steam in sufficient quantities to sustain this pull. This critical speed is different for almost every type and design of locomotive. Fig. 2 may illustrate the point.

Under the best track conditions the locomotive referred to in Fig. 2 holds an even tractive power of 38 800 lb. up to the critical speed of 12½ miles per hour, at which point the capacity of the locomotive boiler reduces the tractive power with increased speed along the curve marked "Cylinder Power at Rim of Drivers."

In regard to freight-train resistances, Mr. Bissell's curve of decrease in resistance due to increase in lading is most interesting, and it would seem that more data might be of value in the matter.

On Fig. 3 the writer has plotted some values, as determined by dynamometer-car tests by Mr. D. C. Crawford in 1901 and by Mr. M. H. Wickhorst in 1904; also a formula proposed by Mr. G. R. Henderson in his "Locomotive Operation," presumably from observed data. These tests were all stated to be for slow speeds and straight

track. Mr. Crawford's and Mr. Wickhorst's figures check remarkably well, and would lead to the conclusion that Mr. Bissell's figures are rather high. Mr. Henderson's formula, however, gives values which come somewhat closer to Mr. Bissell's for heavy loads. The effect of temperature on this curve is not discussed; presumably the values are for conditions above the freezing point. Mr. Riley.

In Equation 2 the question of the definition of tractive power arises. Mr. Bissell calls P the tractive power at the drawbar. From a mathematical and exact point of view, the drawbar pull is a most unsatisfactory figure. The train may be considered as a single body on which a force is impressed, counterbalanced by certain resistances. The force impressed in this case happens to be applied

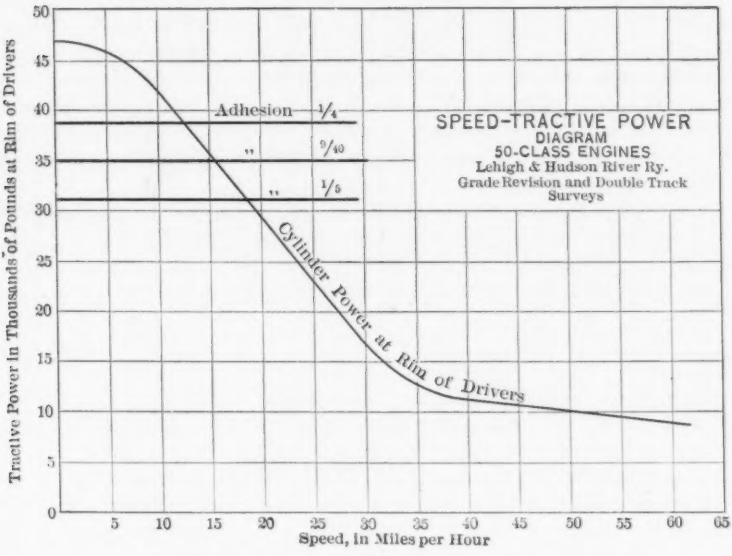

FIG. 2.

by a steam engine mounted on what may be considered one car of this train, but if the engine had been mounted on a concrete base at the summit of the grade, and had exerted its tractive power by a cable, the principle would have been the same. If the engine mounted on the car and the one on the ground were of the same size, design, and steam pressure, the tractive power of each would be the same.

According to Mr. Bissell's formula, however, the tractive power of these two engines would be quite different. There are two balanced forces under consideration, the pull of the prime mover and the resistance of the body moved; the body moved is the total train, not the train back of the engine. The latter consideration will certainly

Mr. Riley. lead to confusion in studying velocity grades. If the train is accelerated or retarded, the drawbar pull, as calculated by Mr. Bissell, is not affected by such acceleration or retardation; but it is obvious that the change in speed does affect the drawbar pull, as the mass of the engine and tender is either absorbing or giving up energy due to its change in momentum. The tractive power of the prime mover, however, is not affected by this change in speed, and the gain or loss in momentum takes its proper place with the other resistances of the body moved.

It has been advanced as an argument against "Cylinder Tractive Power" or "Tractive Power at Rim of Drivers," that an engine with a boiler pressure of 10 or 20 lb. could not move itself, and that it would be ridiculous to assume that it had any tractive power whatsoever. Nevertheless, that engine has just as real a tractive power as the engine which is stalled with its train on a grade. It has a tractive power even though that tractive power is not sufficient to move the mass to which it is applied. What would become of the drawbar pull, as determined in Mr. Bissell's formula, if an attempt were made to apply it to multiple-unit electric trains? One would be simply forced to consider the tractive power of the motors as applied at the wheels and not at the drawbar.

For these reasons it seems to be more logical to modify Mr. Bissell's Equation 2 in some such way as the following:

$$\text{Tons of train weight behind tender} = \frac{T - (20g + l)\, m}{\dfrac{a}{b + w} + 20g}$$

in which

T = Tractive power at rim of drivers,

l = Frictional resistances of locomotive wheels and trucks considered as a loaded car.

The other symbols are the same as Mr. Bissell proposes.

The relation between drawbar pull and tractive power is then as follows:

Drawbar pull, $P = T - (20g + l)\, m$ for constant speed and straight track;

or, to make the formula more universal:

$$P = T - \left[20g + c + l + \frac{1000\,(v_2^2 - v_1^2)}{dg}\right] m$$

in which

c = resistance due to curvature, in pounds per ton of engine weight,

v_1 = velocity, in feet per second, at beginning of period considered,

v_2 = velocity, in feet per second, at end of period considered,

Mr. Riley.

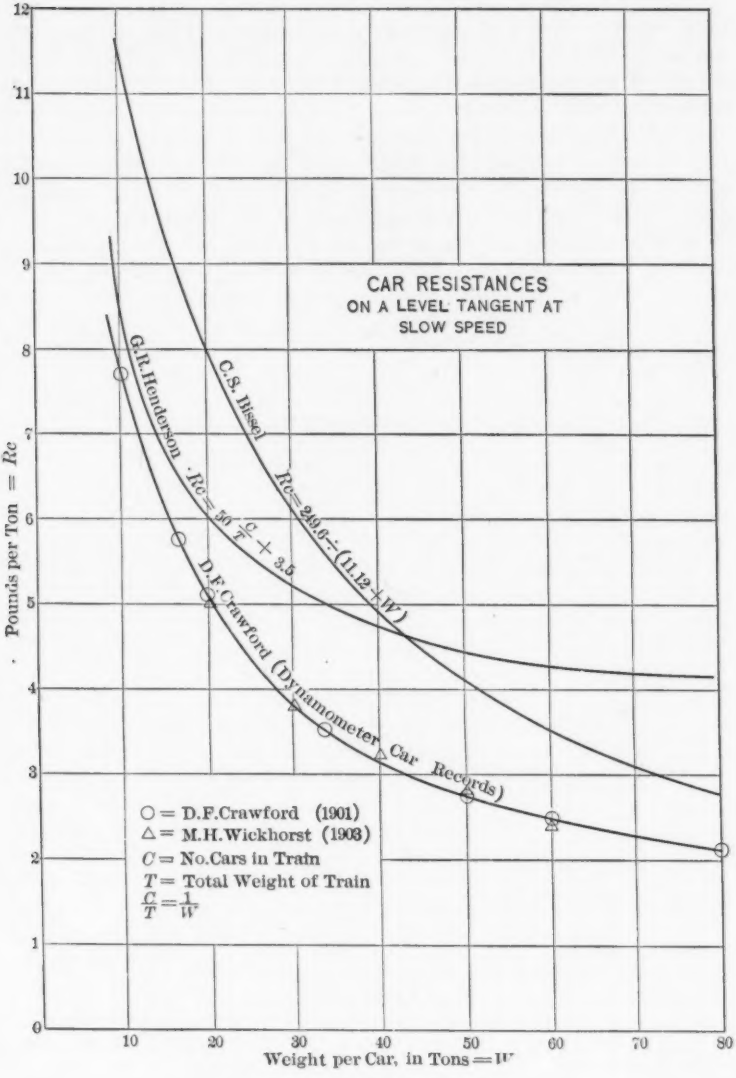

FIG. 3.

Mr. Riley.

d = distance, in feet, covered during period,
m = weight of engine, in tons,
g = acceleration of gravity.

It must be admitted, however, that even this is not exact, since it omits the consideration of the power absorbed or given up by the change in rotative speeds of the engine wheels. However, since it concerns itself only with the engine, it is believed that it is sufficiently exact for all grade-revision purposes excepting possibly the calculation of limiting momentum grades, in which case the methods adopted by Mr. J. B. Berry, in the Union Pacific grade-revision work, are recommended as most exact and logical. A detailed discussion of these methods may be found in the *Proceedings** of the American Railway Engineering and Maintenance-of-Way Association.

* Vol. V.

Vol. XXXV. MAY, 1909. No. 5.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

NICKEL-STEEL EYE-BARS FOR BLACKWELL'S ISLAND BRIDGE.

Discussion.*

By Gustav Lindenthal, M. Am. Soc. C. E.

Gustav Lindenthal, M. Am. Soc. C. E. (by letter).—The writer desires to add some historical information to this paper. Mr. Lindenthal.

The use of large nickel-steel eye-bars in the construction of the Blackwell's Island Bridge, in New York, originated with the writer when he was Commissioner of Bridges (1902-03).

The cross-sections of the tension members were so large that the use of higher steel for them appeared expedient. Certain tests of nickel-steel eye-bars, 6 to 10 in. wide, made at Pencoyd in 1902, showed that a material some 50% stronger than the usual structural steel, and perfectly reliable for forged work, could be obtained.

At first, eye-bars 18 in. wide and from 2 to 2½ in. thick were intended. The existing eye-bar plants could make bars only up to 16 in., but the large tonnage (17 000 tons) contemplated in the Blackwell's Island cantilever structure and in the chain design for the Manhattan Bridge would have justified the erection of a larger plant for the heavier bars.

The writer was not able to obtain the necessary appropriation for both bridges, but only for the Blackwell's Island Bridge. Because of the smaller tonnage to be contracted for, the largest eye-bars were limited to 16 in., the capacity of the existing plant.

* This discussion (of the paper by William R. Webster, M. Am. Soc. C. E., printed in *Proceedings* for March, 1909), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

Mr. Lindenthal.

The attempts, mentioned by Mr. Webster, of substituting riveted eye-bars for the forged nickel-steel eye-bars, were made after the writer had relinquished charge of the work; so also other changes in the design which resulted in the weak structure, which, from the official investigation and reports made after its completion, Blackwell's Island Bridge (now called Queensboro Bridge) is known to be.

Vol. XXXV. AUGUST, 1909. No. 6.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS AND DISCUSSIONS

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THE USE AND CARE OF THE CURRENT METER, AS PRACTICED BY THE UNITED STATES GEOLOGICAL SURVEY.*

By John C. Hoyt, Assoc. M. Am. Soc. C. E.

To be Presented September 15th, 1909.

Introduction.

A current meter for measuring the velocity of flowing water comprises two essential parts: (*a*) a wheel arranged so that when suspended in flowing water the pressure of the water against it causes it to revolve; (*b*) a device for recording or indicating the number of revolutions of this wheel. The relation between the velocity of the moving water and the revolutions of the wheel is determined by rating each meter.

The earliest type of meter was the float wheel, which was used by Borda and Dupuit in the latter part of the eighteenth century, and was practicable only for measuring velocities at the surface. About 1790, Woltmann modified this wheel so that it could be used beneath the surface, the number of revolutions being recorded by a gear mechanism, which was started and stopped at the beginning and end

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Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

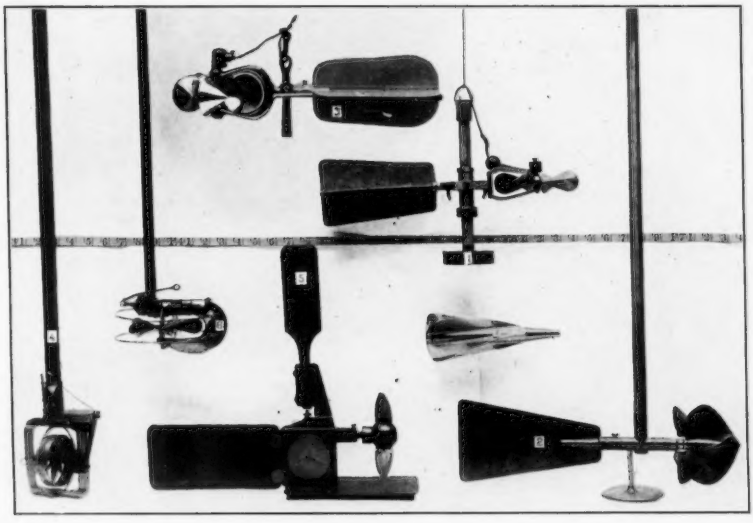


FIG. 1.—TYPES OF METERS EXPERIMENTED WITH BY U. S. GEOLOGICAL SURVEY.

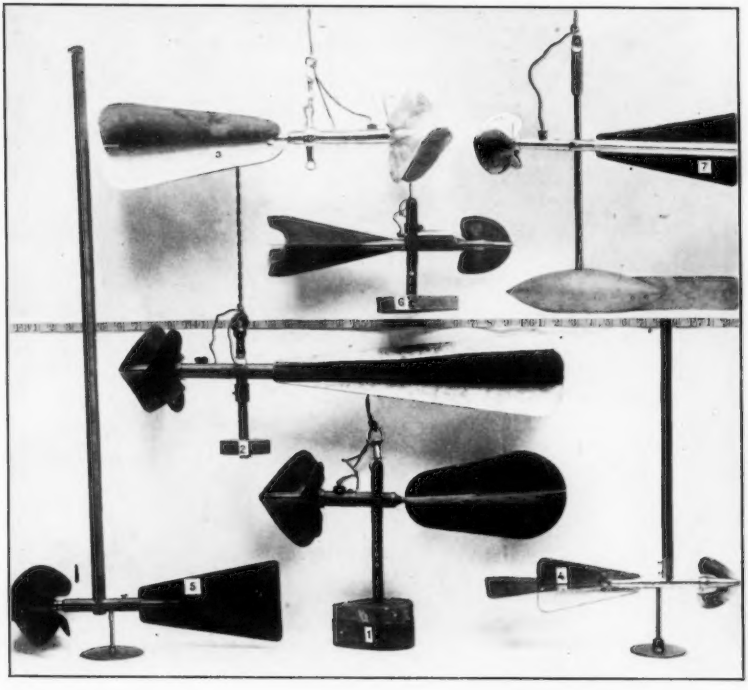


FIG. 2.—VARIOUS FORMS OF THE HASKELL METER.

of a run by a catch operated by a cord. It was necessary, however, to lift the meter out of the water in order to read it. LaPointe arranged the recording apparatus above the surface by connecting the axle with a vertical rod and beveled gear; and Baumgarten, Saxton, Brewster, Laignel, and others made various modifications of the instrument. Prior to the invention of the electric recorder for indicating the number of revolutions of the wheel, users of the meter always experienced difficulty caused by the resistance due to the recording mechanism and its tendency to get out of order.

In America current meters were earliest used in connection with the investigations of the Mississippi, started in 1850 by Humphreys and Abbot,* when the ship's log and the Saxton meter were used to a small extent and with little success.

About 1860, the late D. Farrand Henry, M. Am. Soc. C. E., Assistant on the United States Lake Survey, invented for use with the current meter an electrical recorder,† which eliminated the frictional and the other difficulties of the mechanical recorder, and made feasible the further development of the meter.

The first extended and successful series of measurements with the current meter in the United States was made on the Connecticut River by the late T. G. Ellis, M. Am. Soc. C. E., in connection with studies begun in 1871.‡ General Ellis started his work with the Woltmann meter, equipped with an electrical recording device, but later used an electrical recording meter devised by himself. The results obtained by these measurements have had an important effect on the development of stream-gauging instruments.

The earliest American patents for current meters were taken out in 1851. There are now on file in the Patent Office, classified under ship's logs, more than fifty patents for devices for measuring the velocity of water, and many other devices have been constructed but not patented. The only ones which have had much general use, however, are the Price, Haskell, Fteley, and Ellis meters (Plates XXI, XXII, and XXIII, and Fig. 1).

Each of the various meters has been developed to meet the requirements of some special condition, and, until recently, the use of all

* "Report upon the Physics and Hydraulics of the Mississippi River," 1861.

† *Journal* of the Franklin Institute, Vol. XCII, 1871.

‡ Report, Chief of Engineers, U. S. Army, 1878, Part I.

has been confined to special hydrographic investigations in connection with some public work, municipal, State, or Federal. The present widespread interest in hydraulic development has created such a demand for measurements of the flow of streams that the current meter is now in general use, and is essential to the equipment of the engineer engaged in hydraulics.

In 1888 the United States Geological Survey began the gauging of streams. The plans called for measurements in all sections of the country, and the conditions to be provided for presented an infinite variety in combinations of range in depth, width, and velocity. No adequate meter was available for work of this extended nature, and, furthermore, elaborate equipment and methods were prohibited by limited funds. It was necessary to devise or adapt an instrument which could be readily carried in the field and operated by one man, either from a bridge, a boat, a cable and car, or by wading.

After experimenting with various types (Plates XXI and XXII), the engineers of the Survey developed a meter combining the essential features of the Price acoustic and large Price electric meter (Plate XXII). This is known as the small Price meter, and has been in general use in the Survey work. Modifications in its construction have been made from time to time, based on the experience of various engineers, until now it represents the experience of more than twenty years of stream gauging (Plate XXIII).

The methods developed by the Survey engineers are believed to represent the best practice in this line of work. The Survey's data are now used extensively in hydraulic development in all parts of the country, and its methods have been adopted in similar work by many engineers in all parts of the world. A description of the instruments and their use, therefore, should be of interest to all engineers.

General Features of Current Meters.

Current meters, in general, may be divided into two classes: direct action and differential action, the division depending on whether the water, in revolving the wheel, does or does not exert a force which tends to retard the motion of the wheel.

The wheels of the direct-action meters consist of flat or warped-surface vanes, set on a horizontal axis, which revolve by the direct pressure of the water against the blades. In a meter of this type the

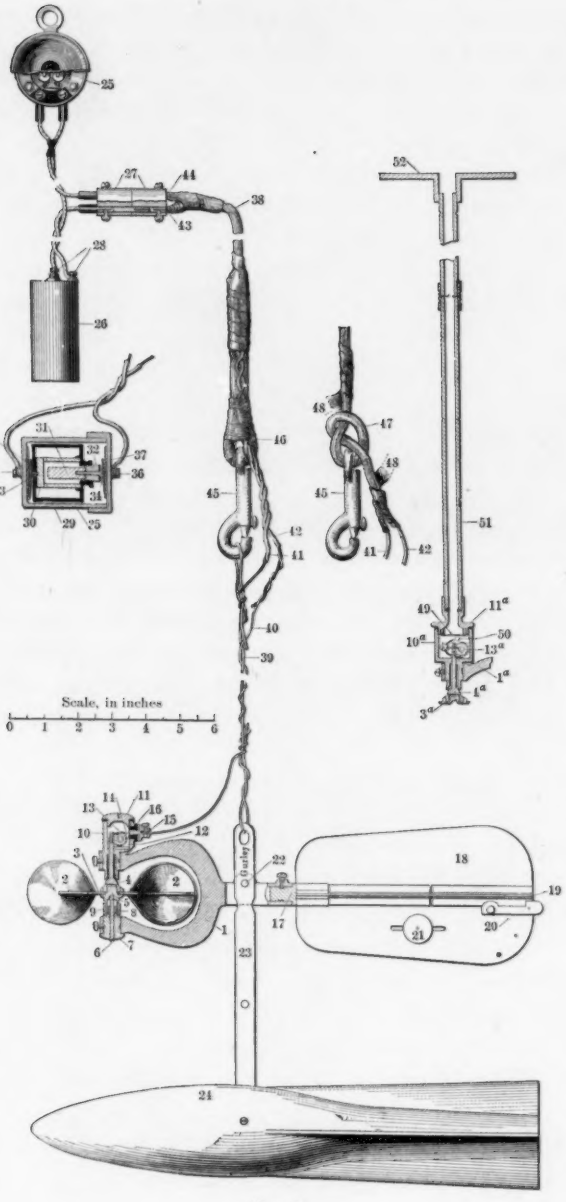


FIG. 1.

friction increases as the velocity decreases. The revolutions of the wheel are quite rapid for high velocities, but the resistance due to the area of the cross-section of the meter against the water is small. The principal direct-action meters are the Haskell and the Fteley (Plate XXI).

The wheels of the differential meters are made up of a series of cups which revolve on a vertical axis. The pressure of the water on the cups, being less on the convex than on the concave side, causes the wheels to revolve. In a meter of this type, the friction increases with the velocity, but the increased frictional effect is overcome by the increased motive power due to the velocity. The area of the cross-section offers a larger resistance to the current than in the direct-action meter, but the wheel revolves more slowly owing to the retarding motion caused by the resistance on the convex side of the cups. The principal types of differential meters are the Price and the Ellis (Plates XXI and XXII).

The essentials for a good current meter are (*a*) simplicity in construction, with no delicate parts which easily get out of order; (*b*) a small area of resistance to the velocity of the water; (*c*) a simple and effective device for indicating the number of revolutions of the wheel; and (*d*) easy adaptability for use under all conditions.

Fig. 1 and Plates XXIV and XXV illustrate the difference in construction between meters of American and foreign make. The foreign meters are much more delicately constructed than the American, and their mechanism is more complicated, so that several men are needed to make a discharge measurement (Fig. 1, Plate XXVI) instead of one man, as with American meters (Plate XXVII).

Although the small Price meter is the only one described herein, an effort has been made to adapt the general discussion to the use of current meters of any make.

Description of the Small Price Current Meter and Equipment.

The small Price current meter (Fig. 1) and equipment consists of five principal parts: (1) the head; (2) the tail; (3) the hanger and weights; (4) the recording device; and (5) the suspending device. In the following descriptions the numbers in parentheses refer to those on Fig. 1.

The Head.—The head consists of a ⊂-shaped yoke (1) carrying

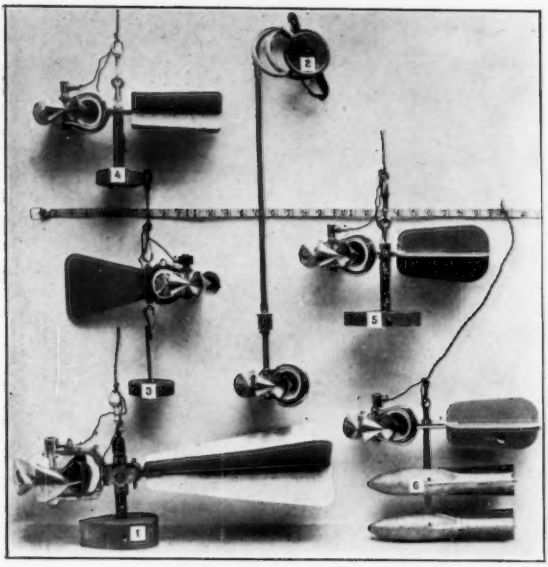


FIG. 1.—VARIOUS FORMS OF RECORDING DEVICES.

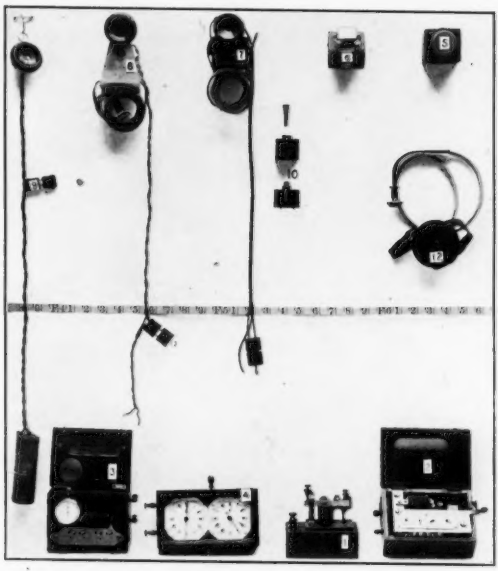


FIG. 2.—VARIOUS FORMS OF THE PRICE METER.

a wheel made of six conical cups (2), brazed to a horizontal frame (3). This wheel, referred to as the cups, turns on a vertical axis known as the cup shaft, which sets on a cone point bearing at the lower end and engages the recording mechanism at the upper end.

The cup shaft consists of two parts (4 and 5), screwed together from either side of the cup frame, thus fastening the cups rigidly to the cup shaft. At the lower part of the cup shaft there is a cone bearing which receives the cone point (6) on which the cups revolve. The cone point is screwed through a metal bushing (7) known as the cone plug, and is firmly held by a lock-nut (8). The cone plug fits into the lower arm of the yoke by a sliding connection, and is clamped in position by a set-screw. By means of a sleeve-nut (9) on the lower part of the shaft, the cups can be lifted from the cone point when the meter is not in use. This sleeve-nut has a left-handed thread, so that it will not tighten when the cups revolve. The upper part of the cup shaft is fitted with a worm gear which passes into a cylindrical chamber (10 or 10^a), known as the contact chamber, as it contains the mechanism for making the contact which indicates the revolutions of the cups. The construction and arrangement of both the contact chamber and the mechanism contained in it depend on whether an acoustic or an electric recording device is used.

When the electric recording device is used, the contact chamber (10) is closed by a screw cap (11) provided with a leather gasket for keeping out the water. It fits by a sliding connection into the upper end of the yoke, and is clamped into position by a set-screw. In the contact chamber there is fitted a cylindrical plug (12) which is held in position by a screw and carries a gear-wheel (13) which engages a worm gear on the upper end of the cup shaft, the gearing being arranged so that the wheel makes one revolution for every twenty revolutions of the cups. On the side of the wheel there are four platinum pins, equally spaced and set so that they will strike the contact spring (14) at each fifth revolution of the cups, thus closing the electric circuit, which is indicated by the recording device, as explained later. These contact parts are known as the contact wheel, the contact pins, and the contact spring. The contact spring is of platinum, and is carried by the contact plug (15) which is screwed into the contact chamber through a hard-rubber bushing (16), thus insulating the contact spring from the meter when it is not touching

one of the pins on the contact wheel. In the end of the contact plug there is a hole and a set-screw for connecting with a wire from the recording device.

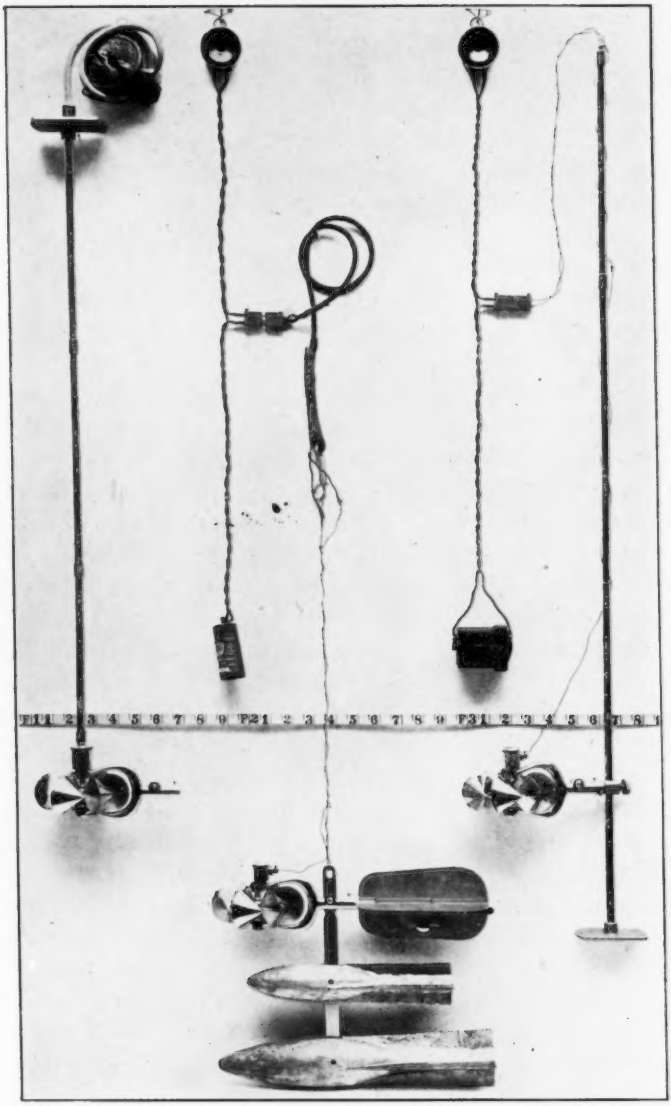
When the acoustic recording device is used, the contact chamber (10^a) is closed with a cap (11^a) and a metal drum (49), and, in place of the platinum contact spring (14) and the plug (16), there is a small hammer (50) which is arranged so that it is thrown against the drum by the pins on the side of the gear-wheel (13^a) at each fifth revolution of the cups.

The stem of the yoke contains a slot and a screw hole (22) for attaching the meter hanger (23), and a socket into which the stem of the tail of the meter (17) is fastened.

The Tail.—The tail is used when the meter is suspended by a cable. It provides for balancing the head, and also keeps the axis of the meter parallel to the direction of the current. It consists of a stem (17) which fits by a sliding connection into the socket in the stem of the yoke where it is clamped by a set-screw. On this stem there are two vanes (18 and 19) set at right angles. One of the vanes is rigidly attached to the stem; the other fits into it by grooves, so that it can readily be pulled out when the key (20) which holds it in place is turned. On one of the vanes there is a slot carrying a weight (21) which can be adjusted so as to balance the meter.

The Hanger and Weights.—When suspended with a cable, the meter is hung by a screw-bolt (22) on a steel stem (23) which passes through a slot in the stem of the yoke. The slot in the stem of the yoke is wide enough to allow the meter to swing freely in a vertical plane, and the bolt passes through the frame a little above the center of gravity of the meter, so that the latter will readily adjust itself to a horizontal position. In the upper end of the hanger there is a hole for attaching the suspending cable, and at intervals along the stem there are other holes by which the meter and lead weights may be hung. The weights (24) are of torpedo shape—this design offering the least resistance to the current—and are made in two sizes, weighing, respectively, 6½ and 15 lb. They are attached to the stem by a screw bolt. The manner of arrangement of the weights and meter on the stem depends on the conditions under which the measurements are to be made.

When the meter is used on a rod, the tail, hanger, and weight are



PRICE METERS AS NOW USED.

dispensed with, and the head is held by a sliding carrier (Plate XXIII), or it is attached to the rod (51) by a screw connection on the contact chamber when the acoustic recording device is used.

The set-screws for clamping the various sliding connections are all of the same size and of standard make. Beveled grooves are provided in each of these connections so that when the set-screws engage them the parts are drawn into place.

All parts of the meter are standard, and can readily be replaced in the field.

The Recording Device.—A recording or indicating device is necessary for determining the number of revolutions of the meter wheel, and the successful use of the meter depends primarily on this part of the apparatus.

Various devices, Fig. 1, Plate XXII, operated either on the mechanical, electric, or acoustic principle, have been used for this purpose. These include the telegraph ticker, electric buzzer, telephone receiver, drums, etc. Of these, however, the telephone attachment and the acoustic recorder have been found to be most satisfactory in general practice.

The telephone attachment, Fig. 1, consists of a telephone receiver (25) and small battery (26), connected in a circuit which terminates in a connecting plug (27) by which the apparatus can be readily connected or disconnected in circuit with the meter, depending on the method of suspension, as explained later.

The magnets of the telephone receiver are wound for 10-ohm resistance so as to make as loud a click as possible.

Either a dry-cell or a wet-cell battery may be used. The most satisfactory dry cell (26) is the No. N 118, "Every Ready" cell, which is 1 in. in diameter and 3 in. long. Connection is made with this cell through two screw connecting posts (28), both at the same end of the cell.

The wet cell consists of an outer casing of hard rubber (29), about 1½ in. square, containing a carbon compartment (30) into which a zinc pole (31) having a rubber stopper (32) is inserted. The current is generated by a solution of bisulphate of mercury and water. Contact is made from the cell through a platinum plug (33) extending into the carbon at the bottom and through the screw (34) in the zinc pole which extends through the rubber stopper.

The cell is encased in a leather box (35), and connection is made

with it through two screw connecting posts (36), each of which terminates in a separate spring plate (37) against which the poles of the battery bear.

In use, the receiver is pinned to the left shoulder and the battery cell is placed in the side coat pocket. The connecting plug (27) will then hang a little below the shoulder and is easily accessible for attaching and detaching the meter.

In the acoustic recorder, the striking of the hammer (50) on the drum (49) in the contact chamber (10^a) indicates each fifth revolution of the meter, as already explained. The sound is transmitted through the rods (51) and a rubber tube to the ear of the observer. The rubber tube and ear-piece (Plate XXIII) are not necessary unless there is considerable noise at the station, as the click is loud enough to be heard without them.

Automatic recorders, Fig. 1, Plate XXII, have been used to some extent to determine the number of revolutions of the meter. For general work, however, these are not entirely satisfactory, as they are likely to get out of order, and are inconvenient to carry.

The time taken by the meter wheel to make a given number of revolutions should be determined with a stop-watch, as by this means the error in the observation can be reduced by observing to a fractional part of a second.

The Suspending Device.—The suspending device, which consists of a rod or of some form of cable, must make provision for lowering the meter and weight into the water and also for completing an electric circuit between the meter wheel and the recording device.

The rod in common use in connection with the electric recorder consists of a ½-in. tube (Plate XXIII) graduated to feet and tenths. For convenience in carrying, it is made in 1½-ft. sections which screw together. On the bottom of the rod there is a flat foot which keeps it from sinking into the bottom of the stream, and at the top there is a plug for connecting one of the wires from the recording device. The head of the meter is attached to a sliding hanger, which can be moved up and down the rod or clamped in any position. The circuit between the meter wheel and the recording device is made by attaching one of the wires from the recording device to the plug in the top of the rod. The other wire follows down the rod, and is attached to the contact plug of the meter.

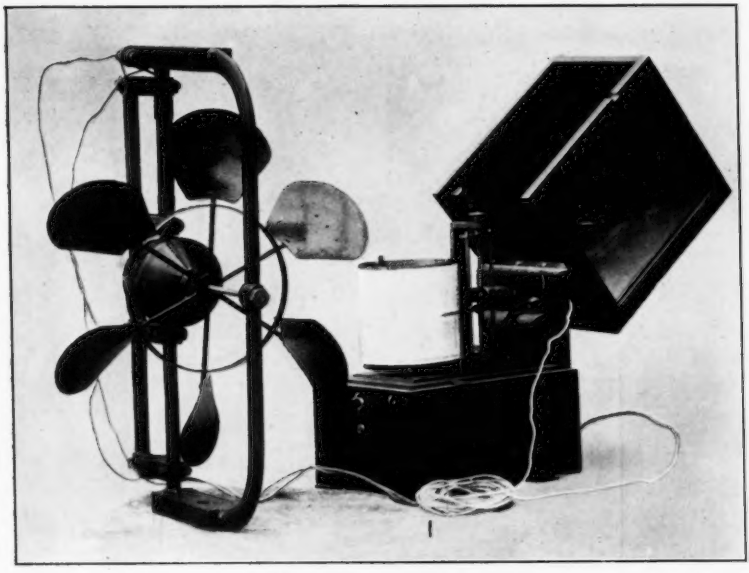


FIG. 1.—RICHARDS METER, WITH ELECTRICAL RECORDER.

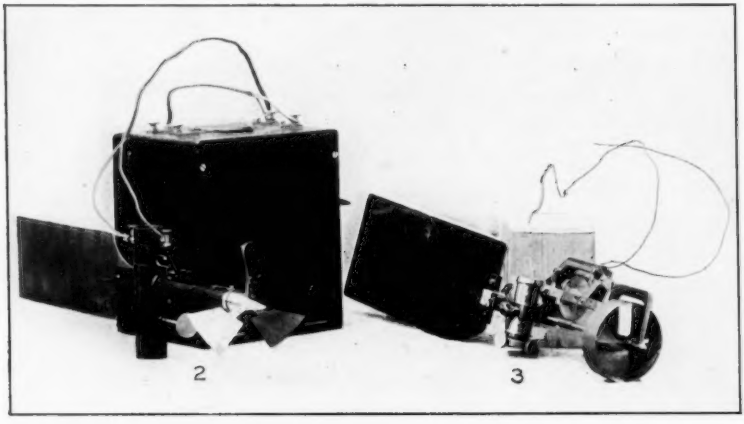


FIG. 2.—OTT AND WOLTMAN METERS, WITH ELECTRICAL RECORDERS.

In using the meter on the rod, the foot-piece of the rod is set on the bottom of the stream, and the head of the meter is adjusted so that it will come to the point at which it is desired to measure the velocity. By this means the meter can be held firmly in the required position.

The rods (Plate XXIII) for use with the acoustic recorder are of ½-in. tubing graduated to feet and tenths, and, for convenience in carrying, are made in 1-ft. sections which screw together. The bottom rod connects with the contact chamber with a screw, and is cut so that the distance from the center of the cups to the end of the rod is just 1 ft. On the upper end of the top rod there is a flat plate, in the center of which there is a hole through which the sound from the drum can be heard. The soundings are made with this end of the rod, and the plate keeps the end from sinking into the bed of the stream.

The best form of cable (Fig. 1) is a combination of No. 16, old code, double-insulated, show-window cord (38) and No. 12 or 14 galvanized wire (39) about which is wound a small insulated magnet wire (40).

The show-window cord is used for the upper part of the cable. It is large enough to be manipulated easily with bare hands, and, being made of two insulated wires, provides for making the direct and return circuit between the meter and the recording device. In its use, the two wires of which it is made must be separated at either end (41, 42, 43, 44) in order to make the attachment with the connecting plug (27) of the recording device at the upper end and with the galvanized wire (39) and magnet wire (40) which lead to the meter at the lower end. A ring or snap (45), into which the galvanized wire is looped, is fastened, either by a loop (46) or a knot (47), to the lower end.

In fastening the meter cable to the snap with a knot (47), a strip of adhesive tape is wound around the cable two or three times, about 1 ft. from the end, leaving about 6 in. of the tape at the beginning and end of the winding. The cable is then inserted through the snap or ring so that the snap bears on the adhesive tape, and a knot is tied in the cable about the snap (45) and drawn down as tight as possible. The ends of the adhesive tape (48) are then wound around the cable, one above and the other below the knot, to keep it from sliding. The outside covering of the cable can then be taken off

within 3 or 4 in. of the knot, and one wire fastened to the wires leading to the contact plug and to the hanger.

If the snap is held in a loop (46), a length of about 12 or 14 in. of the outside insulation is removed so that the wires can be doubled back and connected with those leading to the contact plug and hanger. The loop is first tied with string and then wound with adhesive tape, the tape being placed around the cable where the ring bears on it.

The galvanized and magnet wires (39 and 40) make up the lower end of the cable, and should be long enough to reach from the surface to the deepest point in the stream. Their use is advantageous because they offer small resistance to the moving water and thus prevent the meter from being carried down stream.

The galvanized wire (39) provides both for carrying the weight of the meter and conducting the electric current between the meter and the recording device. It is attached by ordinary loop connections to the snap in the lower end of the show-window cord and to the meter hanger (23). The direct circuit is made through it by the direct connection with the meter stem and by its connection at the upper end with one of the insulated wires (41) from the show-window cord.

The fine magnet wire (40), which provides for the return circuit between the meter and the recording device, is wound loosely around the galvanized wire, and may be fastened with tire tape. At the upper end it is connected with one of the insulated wires (42) from the show-window cord, and at the lower end with the contact plug (15) of the meter. In order to aid in preserving the insulation between the galvanized and magnet wires they should be shellacked; and, even with this precaution, it may be necessary to replace the magnet wire frequently.

If the velocities are not so great as to carry the meter down stream, the galvanized and fine magnet wire may be dispensed with. The snap (45) at the lower end of the show-window cord may be attached directly to the meter stem, the circuit being completed by attaching the insulated wires (41, 42) to the contact plug and to the screw on the meter hanger, respectively.

The meter may also be suspended by a single uninsulated galvanized wire, the return circuit being made through the ground and water. In using the single wire the connection is from the water to the meter through the contact point to the line, then to the battery and through the 'phone to the bridge or cable, then to the ground and back

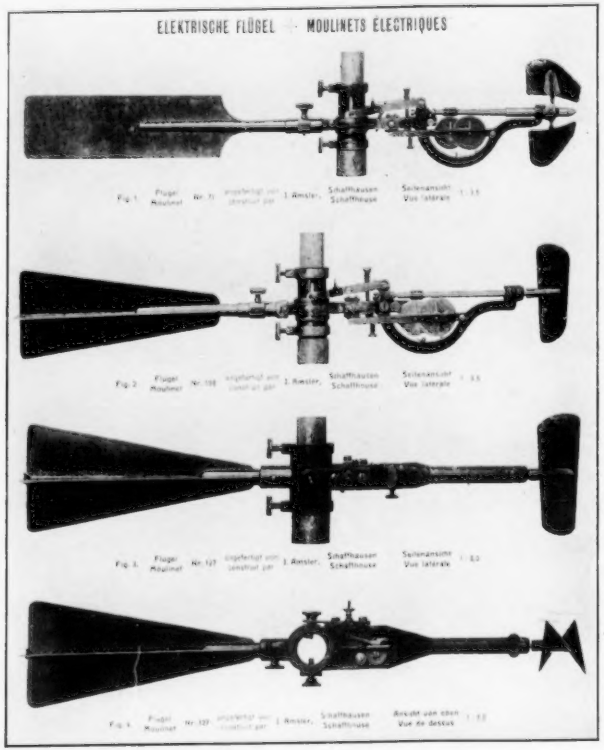


FIG. 1.—TYPES OF AMSLER METER USED IN THE SWISS HYDROGRAPHIC SERVICE.

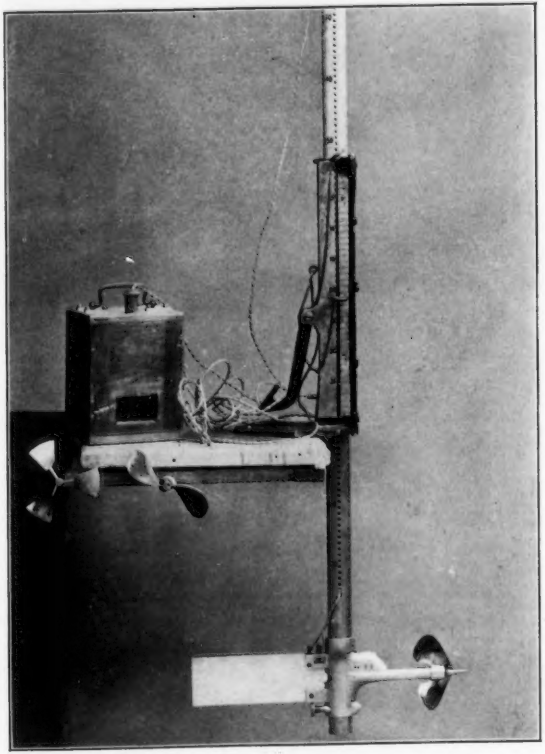


FIG. 2.—OTT METER USED IN THE FRENCH HYDROGRAPHIC SERVICE.

to the water. It does not make any difference on which side of the battery the 'phone is placed in the line (Fig. 2).

When using a single wire, on a cable or a bridge, a clean metallic contact should be made. A little paint, rust, or other coating will prevent good work.

In measuring high velocities and deep streams, where the suspending cables afford insufficient support, stay-lines or guy-lines are used to keep the meter in place.

Rating the Meter.

The relation between the revolutions of the meter wheel and the velocity of the water must be determined by rating each meter before it is used. Theoretically, the rating for all meters of the same make should be the same, but, as a result of slight variations in construction, and in the bearing of the wheel on the axis at different velocities, the ratings differ.

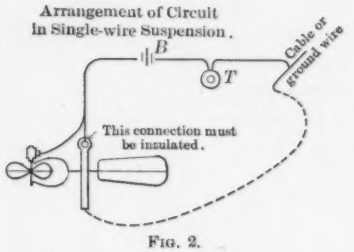


Fig. 2.

A meter is rated by conducting it through still water with uniform speed, and noting the time, the number of revolutions, and the distance. The revolutions per second and the velocity in feet per second are afterward computed from these data. Many runs are made, as shown in Table 1, the speeds varying from the least which will cause the wheel to revolve to several feet per second. The results of these runs, when plotted (Fig. 3) with revolutions per second and velocity in feet per second as co-ordinates, locate the points which define the meter rating curve from which the rating table is prepared, as described later.

The United States Geological Survey has three rating stations, one at Washington, D. C., one at Denver, Colo., and one at Los Angeles, Cal. (Fig. 2, Plate XXVI). These stations consist of a platform over the water carrying a track about 200 ft. long, on which a car for carrying the meter is run. The track is near the edge of the platform, and the meter is suspended in the water by a projecting arm. At the Denver and Washington stations the car is propelled by hand; at the Los Angeles station it is moved electrically.

The number of revolutions of the meter wheel are indicated on an electric recorder; the distance is obtained by an electrical mechanism, which is in circuit with the meter wheel, so that the exact distance for a given number of revolutions is obtained; and the time is taken by a stop-watch.

TABLE 1.—OBSERVATIONS FOR SMALL PRICE PENTA METER RATING. MADE DECEMBER 14TH, 1908, AT CHEVY CHASE LAKE, MARYLAND, BY HENSHAW AND WALTERS. METER HUNG WITH ONE 15-LB. AND ONE 6½-LB. TORPEDO WEIGHT.

No. of run.	Observations for Length of Run.			Time, in seconds.	No. of revolutions.	Revolutions per second.	Velocity, in feet per second.	Remarks.
	Start.	End.	Distance.					
	Feet.	Feet.	Feet.					
1	−6.8	32.0	25.2	79	10	0.127	0.319	
2	21.4	+2.9	24.3	65	10	0.154	0.374	
3	−7.3	32.1	24.8	67	10	0.149	0.370	
4	25.0	−0.4	24.6	62	10	0.161	0.397	
5	−10.7	35.2	24.5	45	10	0.222	0.544	
6	28.2	−2.8	25.4	50	10	0.200	0.508	
7	−13.6	62.5	48.9	78	20	0.256	0.627	
8	54.2	−5.1	49.1	77	20	0.260	0.638	
9	−17.1	66.6	49.5	64	20	0.312	0.773	
10	61.4	−12.9	48.5	68	20	0.294	0.713	
11	−8.7	56.2	47.5	48	20	0.417	0.990	
12	52.1	−5.0	47.1	50	20	0.400	0.942	
13	−1.2	96.4	95.2	76	40	0.526	1.252	
14	83.7	+8.7	92.4	72	40	0.556	1.283	
15	+10.0	82.9	92.9	47	40	0.851	1.977	
16	90.0	+1.8	91.8	50	40	0.800	1.836	
17	+2.0	91.6	93.6	35	40	1.14	2.68	
18	90.7	−0.3	90.4	29	40	1.38	3.12	
19	−9.7	100.8	91.1	14.8	40	2.70	6.16	
20	95.9	−6.3	89.6	17.4	40	2.30	5.15	
21	−4.6	95.6	91.0	12.8	40	3.12	7.11	
22	97.4	−6.8	90.6	11.6	40	3.45	7.81	
23	−10.0	101.3	91.3	11.2	40	3.57	8.15	
24	83.7	+7.4	91.1	9.2	40	4.35	9.90	
25	0.0	92.7	92.7	9.0	40	4.44	10.30	
26	98.0	−7.7	90.3	9.2	40	4.35	9.82	
27	+7.5	86.4	93.9	8.8	40	4.55	10.67	
28	94.5	−1.6	92.9	7.8	40	5.13	11.91	
29	−1.2	93.9	92.7	7.4	40	5.41	12.53	
30	100.6	−9.1	91.5	7.8	40	5.13	11.73	
31	+2.4	91.6	94.0	7.0	40	5.71	13.43	
32	106.0	−13.0	93.0	7.2	40	5.56	12.92	
33	+5.6	88.4	94.0	6.2	40	6.45	15.16	
34	101.1	−7.9	93.2	6.6	40	6.06	14.12	

NOTE.—The runs are in pairs, the odd numbers being across the track and the even numbers in the return to the starting point.

Theoretically, the wheel of a differential-action meter, when carried through still water, should revolve as a wheel revolves in passing over the ground. That is, in going a given distance it should make practically the same number of revolutions, regardless of speed. The rating

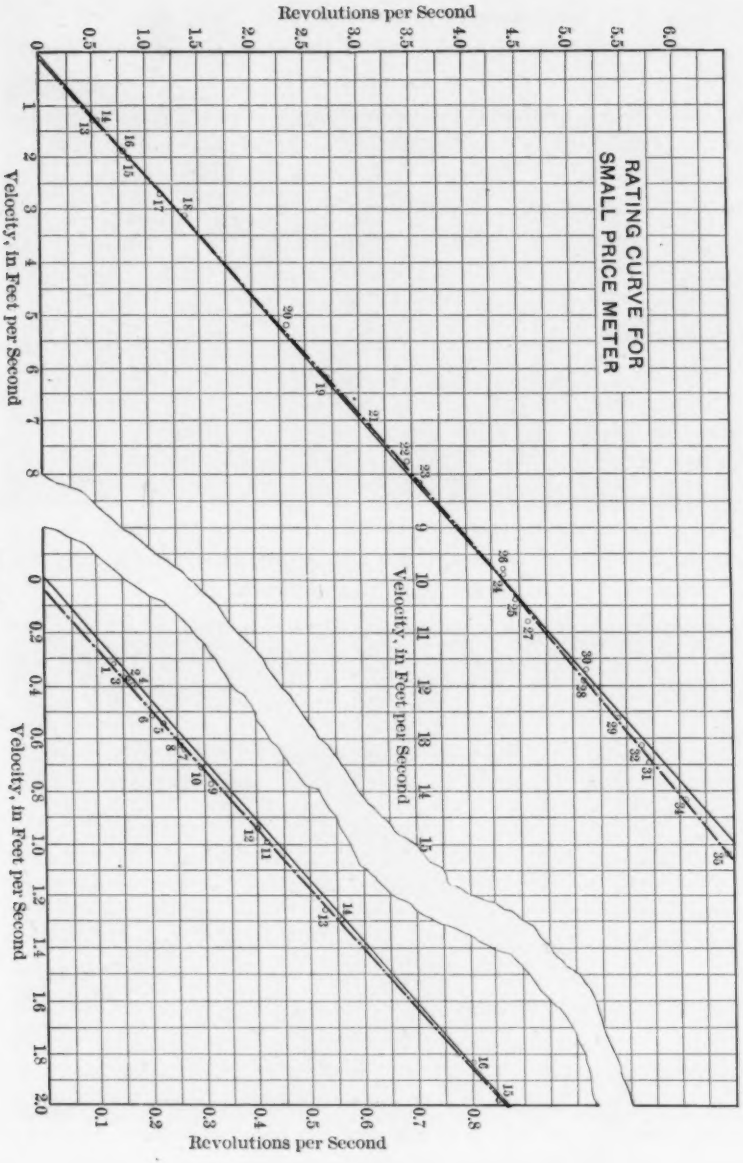

RATING CURVE FOR
SMALL PRICE METER
Revolutions per Second
Velocity, in Feet per Second
Velocity, in Feet per Second
Velocity, in Feet per Second
Revolutions per Second

FIG. 8.

of a great many small Price electric meters shows this number to be from 42 to 44 revolutions in going 100 ft.

The true number of revolutions of the wheel should equal the distance of the run divided by the effective circumference of the wheel multiplied by a coefficient which depends on the retarding effect due to the pressure on the convex surface of the cups and the blanketing effect of the cups. Assuming the effective circumference to be the circle passing through the points of the cups, which is 0.7854 ft., and the true number of revolutions to be 43½ per 100 ft. run; then the coefficient would be 0.345. Although complete data are not available to confirm this theory, the working of the meter shows that it holds very closely to it.

The foregoing shows that the theoretical meter-rating curve is a straight line passing through the origin. If the true number of revolutions made in going 100 ft. is 43½, the equation of this curve will be $X = 2.3\ Y$, where X = velocity, in feet per second, and Y = revolutions per second.

It is interesting to note that Dubuat found that the resistance of a circular plate, when drawn through still water, as compared with that of a sphere of the same diameter, was as 100 to 35.

A study of the rating curves of a large number of small Price meters shows that, as a rule, the curve is made up of two straight lines, the extension of the lower one joining the upper one in an angle between the velocities of 8 and 9 ft. At this point there is a slight increase in the friction on the bearings of the meter wheel and shaft. Notwithstanding this break in the curve, the observed curve parallels the theoretical curve very closely. The lower part of the curve starts at a velocity of less than 0.1 ft. per sec., which is required to start the wheel.

In plotting the rating curve, velocities in feet per second are generally shown as abscissas and revolutions per second as ordinates. The scale for velocities of less than 2 ft. per sec. should be 5 in. per ft. of velocity, and, for greater velocities, 1 in. per ft. of velocity. The scale for revolutions per second, for velocities of less than 2 ft., should be 10 in. per rev. per sec., and for velocities greater than 2 ft., it should be 2 in. per rev. per sec. This necessitates the construction of two curves: one for low velocities and one for high velocities. The low-velocity curve should be constructed first and two points taken from

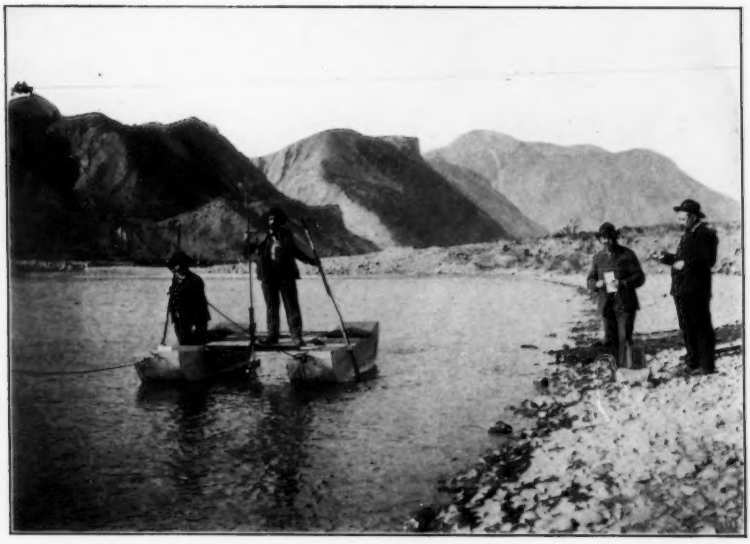


FIG. 1.—MEASUREMENTS AT BOAT GAUGING STATION, ON RIVER DURONÉE, NEAR ROUSSET, FRANCE.

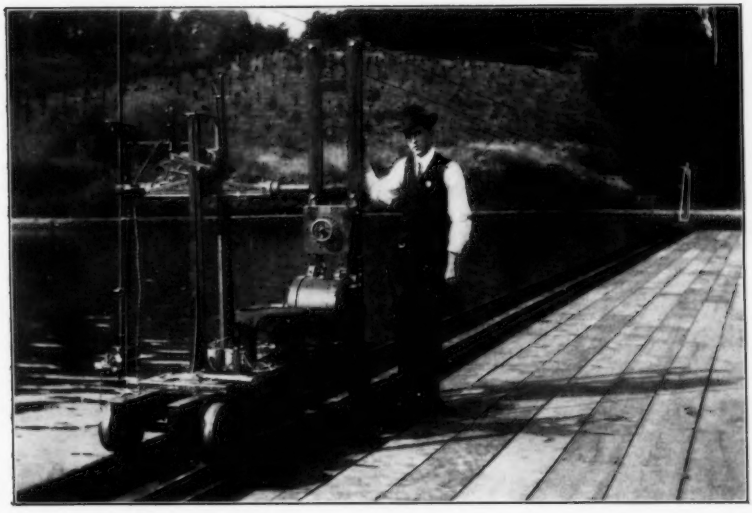


FIG. 2.—U. S. GEOLOGICAL SURVEY CURRENT-METER RATING STATION, LOS ANGELES, CAL.

this curve should be re-plotted at the smaller scale, in order to determine the position of the lower portion of the curve. The theoretical, or a mean, rating curve should also be drawn on the sheet as a guide for drawing the curve.

In using the meter, observation is made of the number of seconds the wheel requires to make a given number of revolutions. Therefore, a rating table is prepared for each meter, giving the velocities per second corresponding to the number of revolutions. The most convenient table is prepared for 5, 10, 20, 30, 40, 50, 60, 70, 80, 90, 100, 150, and 200 revolutions, with the times of the runs ranging from 30 to 60 sec., giving velocities from 0.234 to 15.69 ft. per sec.

The rating table (Table 2) is prepared by tabulating the number of revolutions per second from 30 to 60 sec. for each length of run, as shown in the column headed "Revolutions per second." These values enable taking from the curve, Fig. 3, the velocity per second for each number of seconds run.

As the curves for various meters are practically parallel, differences for consecutive velocities on each are practically the same. Therefore, it is only necessary to take occasional velocities from the curve, and intermediate values can be filled in by adding the successive differences from a mean table, as given in the column headed "Difference." These differences also aid in using the table to interpolate for fractions of a second.

In preparing the mean rating curve and table from which the values in the column headed "Difference" have been obtained, the mean of the ratings of nine meters was taken, the rating table, illustrated by Tables 1 and 2 and Fig. 3, is for a current meter having a rating which is identical with this mean curve and table. Such coincidences often appear, either for the whole curve or for a portion of it.

Table 3 gives the percentage of deviation from the mean curve of the different curves used in determining it. An examination of this table shows that the error introduced, due to differences in the ratings of the meters, is very small, and that for meters in good condition the standard rating table can be used without serious error for all velocities greater than 0.5 ft. per sec. The comparison of numerous other rating curves with the standard shows practically the same percentage of variation.

TABLE 2—*Continued.*

41	0.122	0.322	0.007	0.244	0.599	0.013	0.488	1.15	0.03	0.732	1.71	0.04	0.976	2.26	0.05	1.22	2.82	0.07	2.44	5.60	0.13		41
42	0.119	0.315	0.006	0.238	0.586	0.013	0.476	1.13	0.03	0.714	1.67	0.04	0.952	2.21	0.05	1.19	2.75	0.06	2.38	5.46	0.13		42
43	0.116	0.309	0.006	0.233	0.573	0.012	0.465	1.10	0.02	0.698	1.63	0.04	0.930	2.16	0.05	1.16	2.69	0.06	2.33	5.34	0.12		43
44	0.114	0.303	0.006	0.227	0.561	0.011	0.455	1.08	0.02	0.682	1.60	0.04	0.909	2.11	0.05	1.14	2.63	0.06	2.27	5.22	0.11		44
45	0.111	0.297	0.005	0.222	0.550	0.011	0.444	1.06	0.02	0.667	1.56	0.03	0.889	2.07	0.04	1.11	2.57	0.05	2.22	5.10	0.11		45
46	0.109	0.291	0.005	0.217	0.539	0.011	0.435	1.03	0.02	0.652	1.53	0.03	0.870	2.02	0.04	1.09	2.52	0.05	2.17	4.99	0.11		46
47	0.106	0.286	0.005	0.213	0.528	0.010	0.426	1.01	0.020	0.638	1.50	0.03	0.851	1.98	0.04	1.06	2.47	0.05	2.13	4.89	0.10		47
48	0.104	0.281	0.005	0.208	0.518	0.010	0.417	0.992	0.019	0.625	1.47	0.03	0.833	1.94	0.04	1.04	2.41	0.05	2.08	4.79	0.10		48
49	0.102	0.276	0.005	0.204	0.508	0.009	0.408	0.973	0.019	0.612	1.44	0.03	0.816	1.90	0.04	1.02	2.37	0.05	2.04	4.69	0.09		49
50	0.100	0.272	0.004	0.200	0.499	0.009	0.400	0.954	0.018	0.600	1.41	0.03	0.800	1.86	0.04	1.00	2.32	0.04	2.00	4.60	0.09		50
51	0.098	0.267	0.004	0.196	0.490	0.009	0.392	0.937	0.017	0.588	1.38	0.03	0.784	1.83	0.03	0.980	2.28	0.04	1.96	4.51	0.09		51
52	0.096	0.263	0.004	0.192	0.482	0.008	0.385	0.919	0.017	0.577	1.36	0.02	0.769	1.79	0.03	0.962	2.23	0.04	1.92	4.42	0.08		52
53	0.094	0.259	0.004	0.189	0.473	0.008	0.377	0.903	0.016	0.566	1.33	0.02	0.755	1.76	0.03	0.943	2.19	0.04	1.89	4.34	0.08		53
54	0.093	0.255	0.004	0.185	0.465	0.008	0.370	0.887	0.015	0.556	1.31	0.02	0.741	1.73	0.03	0.926	2.15	0.04	1.85	4.26	0.08		54
55	0.091	0.251	0.004	0.182	0.458	0.007	0.364	0.872	0.015	0.545	1.29	0.02	0.727	1.70	0.03	0.909	2.11	0.04	1.82	4.18	0.07		55
56	0.089	0.247	0.004	0.179	0.450	0.007	0.357	0.857	0.014	0.536	1.26	0.02	0.714	1.67	0.03	0.893	2.08	0.04	1.79	4.11	0.07		56
57	0.088	0.244	0.003	0.175	0.443	0.007	0.351	0.843	0.014	0.526	1.24	0.02	0.702	1.64	0.03	0.877	2.04	0.03	1.75	4.04	0.07		57
58	0.086	0.240	0.003	0.172	0.436	0.007	0.345	0.829	0.013	0.517	1.22	0.02	0.690	1.61	0.03	0.862	2.01	0.03	1.72	3.97	0.07		58
59	0.085	0.237	0.003	0.169	0.430	0.007	0.339	0.816	0.013	0.508	1.20	0.02	0.678	1.59	0.03	0.847	1.97	0.03	1.69	3.90	0.07		59
60	0.083	0.234		0.167	0.423	0.006	0.333	0.803		0.500	1.18		0.667	1.56		0.833	1.94		1.67	3.84	0.06		60

NOTE.—Tabulated values as given above are taken from computations carried to six significant figures. This accounts for the differences in the velocity column not always agreeing in the last place with those given in the difference column.

TABLE 2.—Mean Rating Table for Small Price Meter.

Based on Ratings of Nine Meters, Made During 1908, at Chevy Chase Lake, Maryland.

Equation Curve { Between 0.0 and 3.9 Rev. per Sec. $x = 2.27600y + 0.04400$
" 3.9 " 7.0 " " " $x = 2.44333y - 0.60332$

Time, in Seconds.	5 Revolutions.			10 Revolutions.			20 Revolutions.			30 Revolutions.			40 Revolutions.			50 Revolutions.			100 Revolutions.			Time, in Seconds.
	Revolutions per Second.	Velocity per Second.	Difference.	Revolutions per Second.	Velocity per Second.	Difference.	Revolutions per Second.	Velocity per Second.	Difference.	Revolutions per Second.	Velocity per Second.	Difference.	Revolutions per Second.	Velocity per Second.	Difference.	Revolutions per Second.	Velocity per Second.	Difference.	Revolutions per Second.	Velocity per Second.	Difference.	
30	0.167	0.423		0.333	0.803		0.667	1.56		1.00	2.32		1.33	3.08		1.67	3.84		3.33	7.63		30
			0.012			0.024			0.05			0.07			0.10			0.12			0.24	
31	0.161	0.411		0.323	0.778		0.645	1.51		0.968	2.25		1.29	2.98		1.61	3.71		3.23	7.39		31
			0.011			0.023			0.05			0.07			0.09			0.11			0.23	
32	0.156	0.400		0.312	0.755		0.625	1.47		0.938	2.18		1.25	2.89		1.56	3.60		3.12	7.16		32
			0.011			0.022			0.04			0.06			0.09			0.11			0.22	
33	0.152	0.389		0.303	0.734		0.606	1.42		0.909	2.11		1.21	2.80		1.52	3.49		3.08	6.94		33
			0.010			0.020			0.04			0.06			0.08			0.10			0.20	
34	0.147	0.379		0.294	0.713		0.588	1.38		0.882	2.05		1.18	2.72		1.47	3.39		2.94	6.74		34
			0.010			0.019			0.04			0.06			0.08			0.10			0.19	
35	0.143	0.369		0.286	0.694		0.571	1.34		0.857	1.99		1.14	2.65		1.43	3.30		2.86	6.55		35
			0.009			0.018			0.04			0.05			0.07			0.09			0.18	
36	0.139	0.360		0.278	0.676		0.556	1.31		0.833	1.94		1.11	2.57		1.39	3.21		2.78	6.37		36
			0.009			0.017			0.03			0.05			0.07			0.09			0.17	
37	0.135	0.352		0.270	0.659		0.541	1.27		0.811	1.89		1.08	2.50		1.35	3.12		2.70	6.20		37
			0.008			0.016			0.03			0.05			0.06			0.08			0.16	
38	0.132	0.343		0.263	0.643		0.526	1.24		0.789	1.84		1.05	2.44		1.32	3.04		2.63	6.03		38
			0.008			0.015			0.03			0.05			0.06			0.08			0.15	
39	0.128	0.336		0.256	0.628		0.513	1.21		0.769	1.79		1.03	2.38		1.28	2.96		2.56	5.88		39
			0.007			0.015			0.03			0.04			0.06			0.07			0.15	
40	0.125	0.328		0.250	0.613		0.500	1.18		0.750	1.75		1.00	2.32		1.25	2.89		2.50	5.73		40
			0.007			0.014			0.03			0.04			0.06			0.07			0.14	

TABLE 2—*Continued.*

46	1.30	3.01	0.06	1.52	3.51	0.07	1.74	4.00	0.08	1.96	4.50	0.09	2.17	4.99	0.11	3.26	7.47	0.16	4.35	10.02	0.23	46
47	1.28	2.95	0.06	1.49	3.43	0.07	1.70	3.92	0.08	1.91	4.40	0.09	2.13	4.89	0.10	3.19	7.31	0.15	4.26	9.79	0.22	47
48	1.25	2.89	0.06	1.46	3.36	0.07	1.67	3.84	0.08	1.88	4.31	0.09	2.08	4.79	0.10	3.12	7.16	0.15	4.17	9.58	0.21	48
49	1.22	2.83	0.06	1.43	3.30	0.07	1.63	3.76	0.07	1.84	4.22	0.09	2.04	4.69	0.09	3.06	7.01	0.14	4.08	9.37	0.20	49
50	1.20	2.78	0.06	1.40	3.23	0.07	1.60	3.69	0.07	1.80	4.14	0.08	2.00	4.60	0.09	3.00	6.87	0.13	4.00	9.17	0.19	50
51	1.18	2.72	0.05	1.37	3.17	0.06	1.57	3.61	0.07	1.76	4.06	0.08	1.96	4.51	0.09	2.94	6.74	0.13	3.92	8.98	0.18	51
52	1.15	2.67	0.05	1.35	3.11	0.06	1.54	3.55	0.07	1.73	3.98	0.08	1.92	4.42	0.09	2.88	6.61	0.13	3.85	8.80	0.18	52
53	1.13	2.62	0.05	1.32	3.05	0.06	1.51	3.48	0.07	1.70	3.91	0.07	1.89	4.34	0.08	2.83	6.49	0.12	3.77	8.63	0.17	53
54	1.11	2.57	0.05	1.30	2.99	0.06	1.48	3.42	0.06	1.67	3.84	0.07	1.85	4.26	0.08	2.78	6.37	0.12	3.70	8.47	0.16	54
55	1.09	2.53	0.05	1.27	2.94	0.05	1.45	3.35	0.06	1.64	3.77	0.07	1.82	4.18	0.08	2.73	6.25	0.11	3.64	8.32	0.15	55
56	1.07	2.48	0.04	1.25	2.89	0.05	1.43	3.30	0.06	1.61	3.70	0.07	1.79	4.11	0.07	2.68	6.14	0.11	3.57	8.17	0.15	56
57	1.05	2.44	0.04	1.23	2.84	0.05	1.40	3.24	0.06	1.58	3.64	0.06	1.75	4.04	0.07	2.63	6.03	0.11	3.51	8.03	0.14	57
58	1.03	2.40	0.04	1.21	2.79	0.05	1.38	3.18	0.06	1.55	3.58	0.06	1.72	3.97	0.07	2.59	5.98	0.10	3.45	7.89	0.14	58
59	1.02	2.36	0.04	1.19	2.74	0.05	1.36	3.13	0.05	1.53	3.52	0.06	1.69	3.90	0.07	2.54	5.83	0.10	3.39	7.76	0.13	59
60	1.00	2.32	0.04	1.17	2.70	0.05	1.33	3.08	0.05	1.50	3.46	0.06	1.67	3.84	0.06	2.50	5.73	0.10	3.33	7.63	0.13	60

NOTE.—Tabulated values as given above are taken from computations carried to six significant figures. This accounts for the differences in the velocity column not always agreeing in the last place with those given in the difference column.

TABLE 2—*Continued.*

Time, in Seconds.	60 Revolutions.			70 Revolutions.			80 Revolutions.			90 Revolutions.			100 Revolutions.			150 Revolutions.			200 Revolutions.			Time, in Seconds.
	Revolutions per Second.	Velocity per Second.	Difference.	Revolutions per Second.	Velocity per Second.	Difference.	Revolutions per Second.	Velocity per Second.	Difference.	Revolutions per Second.	Velocity per Second.	Difference.	Revolutions per Second.	Velocity per Second.	Difference.	Revolutions per Second.	Velocity per Second.	Difference.	Revolutions per Second.	Velocity per Second.	Difference.	
30	2.00	4.00		2.33	5.35		2.67	6.11		3.00	6.87		3.33	7.63		5.00	11.61		6.67	15.69	0.53	30
31	1.94	4.45	0.15	2.26	5.18	0.17	2.58	5.92	0.20	2.90	6.65	0.22	3.23	7.39	0.24	4.84	11.22	0.39	6.45	15.16	0.49	31
32	1.88	4.31	0.14	2.19	5.02	0.16	2.50	5.73	0.18	2.81	6.45	0.21	3.12	7.16	0.23	4.69	10.85	0.37	6.25	14.67	0.46	32
33	1.82	4.18	0.13	2.12	4.87	0.15	2.42	5.56	0.17	2.73	6.25	0.19	3.03	6.94	0.22	4.55	10.50	0.35	6.06	14.20	0.46	33
34	1.76	4.06	0.12	2.06	4.73	0.14	2.35	5.40	0.16	2.65	6.07	0.18	2.94	6.74	0.20	4.41	10.18	0.33	5.88	13.77	0.44	34
35	1.71	3.95	0.11	2.00	4.60	0.13	2.29	5.25	0.15	2.57	5.90	0.17	2.86	6.55	0.19	4.29	9.87	0.32	5.71	13.36	0.41	35
36	1.67	3.84	0.10	1.94	4.47	0.12	2.22	5.10	0.14	2.50	5.73	0.16	2.78	6.37	0.18	4.17	9.58	0.30	5.56	12.97	0.39	36
37	1.62	3.73	0.10	1.89	4.35	0.11	2.16	4.97	0.13	2.43	5.58	0.15	2.70	6.20	0.17	4.05	9.30	0.28	5.41	12.60	0.37	37
38	1.58	3.64	0.09	1.84	4.24	0.11	2.11	4.84	0.12	2.37	5.43	0.14	2.63	6.03	0.16	3.95	9.04	0.26	5.26	12.26	0.35	38
39	1.54	3.55	0.09	1.79	4.13	0.10	2.05	4.71	0.12	2.31	5.30	0.13	2.56	5.88	0.15	3.85	8.80	0.24	5.13	11.93	0.33	39
40	1.50	3.46	0.08	1.75	4.03	0.10	2.00	4.60	0.11	2.25	5.16	0.12	2.50	5.73	0.14	3.75	8.58	0.22	5.00	11.61	0.31	40
41	1.46	3.37	0.08	1.71	3.93	0.09	1.95	4.48	0.11	2.20	5.04	0.12	2.44	5.60	0.13	3.66	8.37	0.21	4.88	11.32	0.30	41
42	1.43	3.30	0.08	1.67	3.84	0.09	1.90	4.38	0.10	2.14	4.92	0.11	2.38	5.46	0.13	3.57	8.17	0.20	4.76	11.03	0.27	42
43	1.40	3.22	0.07	1.63	3.75	0.08	1.86	4.28	0.10	2.09	4.81	0.11	2.33	5.34	0.12	3.49	7.98	0.18	4.65	10.76	0.26	43
44	1.36	3.15	0.07	1.59	3.66	0.08	1.82	4.18	0.09	2.05	4.70	0.10	2.27	5.22	0.11	3.41	7.80	0.17	4.55	10.50	0.25	44
45	1.33	3.08	0.07	1.56	3.58	0.08	1.78	4.09	0.09	2.00	4.60	0.10	2.22	5.10	0.11	3.33	7.69	0.16	4.44	10.26	0.24	45

TABLE 3.—Deviation of Velocity in Individual Current Meter Rating Tables from that of the Mean Table, Expressed as a Percentage of the Velocity of the Mean Table.

Revolutions per second.	Velocity, Mean Table.	Meter Number.								
		738	739	788	789	790	792	793	794	795
	Feet per second.	Per cent.	Per cent.	Per cent.	Per cent.	Per cent.	Per cent.	Per cent.	Per cent.	Per-cent.
0.1	0.269	−2.6	−1.5	−2.2	−0.74	−3.3	+7.4	+3.3	−3.0	+0.74
0.2	0.494	+0.40	−1.4	−0.20	−0.81	−1.8	+4.4	+1.8	−1.6	0.0
0.3	0.721	+1.1	−1.8	0.0	−1.5	−1.2	+3.0	+1.4	−1.1	0.0
0.4	0.945	+1.8	−1.6	+0.1	−1.4	−0.95	+2.3	+0.95	−0.95	−0.32
0.5	1.17	0.0	−1.7	0.0	−0.85	−0.85	+1.7	+0.85	−0.85	0.0
0.6	1.39	0.0	−1.4	+0.72	−0.72	0.0	+2.2	+1.4	−0.72	0.0
0.7	1.62	0.0	−1.2	0.0	−0.62	−0.62	+1.2	0.0	−0.62	0.0
0.8	1.84	0.0	−0.54	+1.1	−0.54	+0.54	+1.6	+0.54	0.0	−0.54
0.9	2.07	0.0	−0.97	+0.48	−0.48	0.0	+1.9	+0.97	0.0	0.0
1.0	2.30	−0.43	−0.87	+0.87	−0.43	0.0	+1.1	+1.7	+0.43	−0.87
1.5	3.44	−1.2	−0.58	+0.29	−0.87	−0.29	+0.87	+1.2	+0.29	−0.58
2.0	4.59	−0.87	−0.87	+0.22	−1.3	−0.44	+1.1	+1.1	+0.44	0.0
2.5	5.73	−0.35	−0.70	+0.35	−1.2	−0.35	+1.0	+1.2	+0.52	+1.7
3.0	6.87	−0.15	−1.0	0.0	−1.3	−0.15	+1.0	+1.2	+0.58	+0.44
3.5	8.01	+0.12	−1.0	+0.12	−1.4	+0.75	+0.62	+0.62	+0.37	+0.25
4.0	9.17	0.0	.....	+0.54	−0.11	−0.33	.....	.....	.....	.....
4.5	10.38	−0.48	.....	+0.29	+0.48	−0.10	.....	.....	.....	.....
5.0	11.58	−0.86	.....	+0.34	+0.69	−0.09	.....	.....	.....	.....
5.5	12.82	.....	.....	−0.23	+0.62	−0.31	.....	.....	.....	.....
6.0	14.06	.....	.....	−0.50	+0.71	−0.28	.....	.....	.....	.....
6.5	15.26	.....	.....	−0.52	+0.59	.....	.....	.....	.....	.....

Care of the Current Meter.

Although the current meter is substantially made, and will stand considerable hard usage, it needs special attention and care to insure its proper working.

In taking the meter apart, remove the tail vanes and the hanger stem; then loosen the set-screw to the contact chamber, and pull the chamber out by a slight twisting motion. Care must be taken to let the cups be free to turn, so that the worm gear on the upper end of the shaft can disengage itself from the teeth of the contact wheel. In handling the contact chamber, it is well to take off the cap, so that the gear-wheel can be seen during the operation. The cone point can then be taken out and the cups released by loosening the upper part of the shaft with a spanner wrench. This wrench is arranged so that all parts of the meter can be loosened with it.

In putting the meter together, first attach the cups to the cup shaft. In doing this, the upper part of the shaft should be inserted

through the upper hole of the yoke before it is screwed to the lower part. Care must be taken to place the cups so that they will move counter-clockwise. After the cups have been fastened to the shaft, insert the cone point and clamp it in place, and then insert the contact chamber. In replacing the contact chamber, the cups should be left free to move on the cone point, and care should be taken not to allow the cogs on the worm gear to catch on the teeth of the contact wheel. Before inserting the cone plug, the cone point should be adjusted and firmly secured with a lock-nut. The adjustment should allow a slight vertical motion of the cups.

There are two cases in which the meter can be carried: one is a box, 19 by 7 by 5 in., and the other is a box, 8½ by 6½ by 5 in. The larger box takes the meter with the tail attached, and allows sufficient room for carrying a small weight, a cable for suspending the meter, and miscellaneous tools. The small case is arranged with a shoulder-strap, and is just large enough to carry the meter and the tail when taken apart. The weights, the cable, and other equipment have to be carried in a separate case.

The following articles should accompany each current-meter equipment: A small screw-driver, a pair of parallel pliers with a wire cutter, a spanner wrench, an oil-can, a roll of adhesive tire tape, 25 ft. of metallic tape, a 50-ft. steel tape, an extra cone point, and extra set-screws.

In caring for and using a current meter, the points noted in the following paragraphs should be carefully considered:

1.—Be sure that the set-screws are all tightened before putting the meter in the water; otherwise one of the parts may be lost.

2.—Loosen the sleeve-nut and see that the meter runs freely before beginning a measurement; and spin the meter cups occasionally during a measurement to see that they are running freely.

3.—See that the weights play freely on the stem, so as to take the direction of the current and thus avoid a drag on the line.

4.—If any apparent inconsistency in the results of an observation throws doubt on its accuracy, investigate the cause at once. Grass may be wound around the cup shaft; the cups may be tilted by tension on the contact wire; the channel may be obstructed immediately above the meter; the meter may be in a hole; or the cups may be bent so as to come in contact with the yoke.

5.—After a measurement, clean and oil the bearings (in order to prevent rust) and inspect the cone point.

6.—In packing the meter, turn the sleeve-nut to lift the cups from the cone point.

7.—Always see that the lock-nut on the cone point is firmly screwed against the cone plug.

8.—If the cone point is dulled, it can be sharpened with an oil-stone.

9.—In measuring low velocities, be sure that the meter is in a horizontal position. If it has a tendency to tip, it can be held in place by using a plug in the slot for the stem.

10.—Avoid taking measurements in velocities of less than ½ ft. per sec., as the accuracy of the meter diminishes as zero velocity is approached. Compare velocities of 0.5 ft. per sec. and less, as shown in Table 3.

11.—For velocities of less than 1 ft. per sec., the bearing point should be sharp and smooth, as at the time of rating. As the velocity increases, the condition of the point is less important, for then the friction becomes a small factor.

12.—In taking measurements at high velocities, sufficient weight, or a stay-line, should be used to hold the hanger so that the meter will remain horizontal.

13.—In very shallow streams the meter should be suspended from the lower hole on the stem, and the weight should be placed above.

14.—If the cups of a small Price meter are bent, they may be easily put in shape by using a wood or metal bar with a round, smooth end.

15.—The telephone receiver is very sensitive to electric currents, and can be used to locate any break in the circuit. First try the telephone and battery together (Fig. 4) in a circuit having a make-and-break point, as at *a*. This may be done by using a knife blade or a screw-driver, making connection where the wires enter the plug. If there is no click in the 'phone, then the battery or the 'phone does not make a circuit. If there is a click, insert the meter in the line and test for a contact in the meter head (Fig. 5) by revolving the meter wheel. If the meter is all right, put the meter cord in the circuit and test both sides by making double connection and touching alternate sides of the line, *a* (Fig. 6).

16.—When the meter is not in use, disconnect the meter line from the battery, so that it will not become exhausted.

17.—When a measurement has been completed, the solution may be left in the wet cell if the zinc pole and stopper are replaced by a rubber cork.

18.—Never let the bisulphate dry in the cell, as it forms a hard cake and polarizes the battery.

19.—Do not let any bisulphate of mercury remain loose in the meter box; if it gets into the meter bearings it will corrode them.

20.—The zinc pole in the bisulphate cell sometimes gets pushed down so that it touches the bottom of the cell, in which case the cell is short-circuited and the recorder will not indicate. To test this, lift the plug a little way out of the battery and see if there is a flow of current.

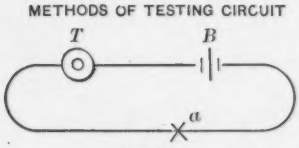


FIG. 4.

21.—Keep the points clean where the battery makes contact with the metal plates.

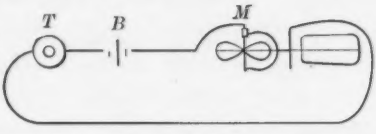


FIG. 5.

22.—The amount of current necessary to work with a telephone receiver is very small, and a battery may be serviceable even though nearly exhausted.

23.—If care is taken, it is very improbable that the telephone receiver will get out of order.

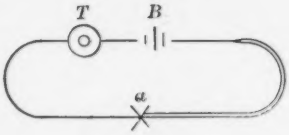


FIG. 6.

24.—Do not strike the telephone receiver, as a heavy jar will to a greater or less extent demagnetize the pole pieces, and to that extent will injure the receiver.

25.—Care must be taken not to short-circuit the dry battery when the meter is not in use, as in that way the cell becomes exhausted in a short time, the energy being used in heating the cell. To avoid this, the poles are wound with adhesive tape.

26.—If a dry cell which has been long in stock fails to work well, punch two nail holes in the wax on top of the cell and put it in water

over night, when it will absorb enough moisture to renew it. The holes should then be coated over by heating the wax with a match and pressing it into place, or by pouring in melted paraffin. A cell which has been exhausted by use is not benefited much by this treatment. The life of a cell depends largely on the amount of leakage in the line during use.

Discharge Measurements with the Current Meter.*

The measurement of stream discharge, for which the current meter was devised, consists in the determination of the quantity of water flowing past a given section of a stream in a given time. This quantity is obtained as the product of two factors: (*a*) The area of the cross-section of the stream, which is determined by sounding; and (*b*) the velocity of the water, which is measured with the current meter. The usual unit for expressing the discharge of a stream is the second-foot—an abbreviation for cubic foot per second—equivalent to the quantity of water flowing in a stream 1 ft. wide and 1 ft. deep, at a velocity of 1 ft. per sec.

Sections at which discharge measurements are regularly made are known as gauging stations, and may be divided into four classes, depending on whether the observations of velocity and depth are made from (*a*) a bridge, (*b*) a cable, (*c*) a boat, or (*d*) by wading (Plates XXVII and XXVIII). The method of making measurements is the same for all stations.

The ideal measuring section should be perpendicular to the thread of the velocity of the water and at a point on the stream where the conditions of bank and bed, both above and below the section, are permanent. This section, Fig. 7, is divided into partial areas bounded by perpendiculars terminating at points in the surface. These are known as measuring points, as the observations of the depth of the stream and the mean velocity of the water in the vertical are made at such points. The distance between the measuring points will depend on the size and character of the stream. The points should be spaced so as to show any irregularities either in the cross-section or in the velocity. If the section is to be used for future measurements, the measuring points should be indicated by permanent marks placed

* A detailed description of the collection and use of stream-flow data and the preparation of estimates of discharge may be found in "River Discharge," by Hoyt and Grover, published by Wiley & Sons, New York.

FIG. 1.—MEASUREMENT BY WADING, GRAND CENTRAL RIVER, ALASKA.

FIG. 2.—MEASUREMENT AT CABLE GAUGING STATION, YAKIMA RIVER, NEAR UNION GAP, WASH.

at intervals of 5 or 10 ft. on the bridge rail or floor of a bridge station, on the cable of a cable station, or on the tape or tagged line usually stretched across a stream when the measurements are made from a boat or by wading.

The procedure in the measurements varies somewhat with the sounding appliance. If the soundings are made with meter and cord, the depth and velocity are usually observed at each successive measuring point across the stream. If a rod or sounding line is used without the meter, soundings are made at all the measuring points before taking the velocities.

TYPICAL CROSS-SECTION OF STREAM
TO ILLUSTRATE DISCHARGE MEASUREMENT

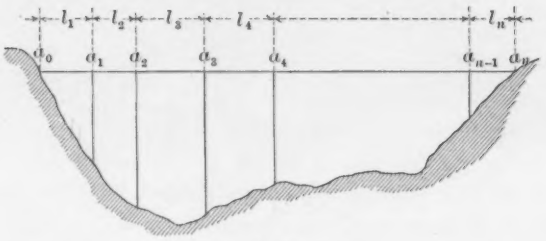


FIG. 7.

In sounding with the rod, care is taken that it does not sink into the bed of the stream, and that the reading is not too high on account of the water running up on it. The soundings from bridges or cables are usually made with the weight and line, and in such cases care is taken to eliminate the error due to the weight being carried down the stream or to the bowing of the line. Soundings with the line are most readily taken as follows: The weight and line are lowered until the weight rests on the bed of the river directly under the measuring point, and, with the line taut, a point is marked on it opposite a fixed point on the bridge or car; the weight is then raised until it just touches the surface of the water and the length of the sounding line that passes the fixed point is measured. The depth is measured by placing the end of a linen or metallic tape opposite the fixed starting point on the sounding line, grasping both the line and the tape in the hands, and drawing up the line and tape without permitting them to slip on each other until the weight reaches the surface of the water. The length of line thus drawn up, representing the depth of the water, is then

read directly from the tape. This measurement can usually be made by one person, even when the water is from 10 to 12 ft. deep.

In making a velocity measurement, the meter is held at the point in the stream at which it is desired to ascertain the velocity of the current. The wheel is allowed to revolve for a few seconds, in order that it may adjust itself to the current, after which the time for a given number of revolutions is noted, from which the velocity is obtained from the rating table for the meter. The time should be about 40 sec., and the number of revolutions will depend on the velocity of the water. A check on the work is made by repeating the observation. If the run is not repeated, a check can be obtained by noting mentally the time for each five revolutions. A stop-watch is used for observing the time, and the record is made to the nearest 0.2 sec., usually the smallest interval indicated with the ordinary stop-watch.

In discharge measurements, the mean horizontal velocity in a vertical at the measuring points is desired. Various methods are used for this determination, among which the following four are most common: (*a*) Vertical velocity-curve method, (*b*) two-tenths-and-eight-tenths-depth method, (*c*) six-tenths-depth method, and (*d*) subsurface method.

(*a*).—By the vertical velocity-curve method, measurements of horizontal velocity are usually made just beneath the surface, at 0.5 ft. below the surface, and at each fifth to each tenth of the depth from the surface to the bed of the stream, and as near the bottom as possible. These measured velocities, when plotted with depths as ordinates and velocities as abscissas, define for each vertical the vertical velocity-curve, which shows the velocity at every point in the vertical, and from which its mean velocity can be determined by dividing the area bounded by the curve and the axis of the ordinates by the mean depth. The following three methods are used in this determination:

1.—Determine the area with the planimeter and divide by the depth.

2.—Divide the area into any number of sections of equal depth, usually ten, and take the mean of the velocities at mid-point of each of these sections as the mean velocity.

3.—Divide the area into sections of convenient depth, which will be equal except for the bottom section, which may have an odd depth.

FIG. 1.—MEASUREMENT AT BRIDGE GAUGING STATION.

FIG. 2.—MEASUREMENT AT BOAT GAUGING STATION.

If the bottom section is the same depth as the others, take the mean of the middle ordinates of each section for the mean velocity. If the bottom section is an odd depth, multiply its mean velocity by the ratio of its depth to that of the common depth of the other sections, and add this product to the sum of the middle ordinates of the other sections, and divide by the number of sections.

Studies of vertical velocity-curves taken on many streams under various conditions of depth, velocity, and roughness of bed, show that these vertical velocity-curves have approximately the form of a parabola in which the axis, coinciding with the filament of maximum velocity, is parallel with the surface and is in general situated between the surface and one-third of the depth of the water. From the maximum the velocity decreases gradually upward to the surface and downward nearly to the bottom, where it changes more rapidly on account of the friction on the bed. As the depth and velocity increase, the curve approaches a vertical line as its limiting position.

The vertical velocity-curve method is valuable as a basis for the comparison of all other methods, for determining the coefficients to be used in reducing the values obtained by other methods to the true value, for use under new and unusual conditions of flow, and for measurements under ice.

(*b*).—In the two-tenths-eight-tenths method, the observations of velocity are taken at two points located at depths from the surface of 0.2 and 0.8 of the depth at the measuring point. The mean velocity is taken as the mean of the velocities at these two points. This method is based on the theory that the vertical velocity-curve is a parabola, as already stated, in which case the mean of the ordinates at 0.2114 and 0.7886 gives the mean ordinate. A study of a large number of vertical velocity-curves shows that this holds true in Nature; and experience proves that this method gives more consistent results than any of the others except the vertical velocity-curve method.

(*c*).—In the six-tenths method, the observation of the velocity is taken at a depth from the surface equal to 0.6 of the depth of the stream. This method is also based on the theory that the vertical velocity-curve is a parabola with the maximum abscissa between zero and one-third the depth, in which case the mean ordinate is between 0.58 and 0.67 of the depth from the surface. A study of a large number of vertical velocity-curve measurements shows that the mean depth

of the mean velocity is approximately 0.6 of the depth. This method has the advantage of ease in making the measurement, and gives very satisfactory results but not as good as those obtained by the two-tenths-eight-tenths method.

(*d*).—In the sub-surface method, the measurement of velocity is made at from 0.5 to 1 ft. below the surface, depending on the depth of the stream. The meter is held at sufficient depth to be out of any surface disturbance. When this method is used, the velocity must be reduced by a coefficient to obtain the mean velocity. This coefficient varies between 78 and 98%, depending on the depth and the velocity of the stream. The deeper the stream and the greater the velocity, the greater the coefficient. For average streams in moderate freshets, a coefficient of 90% should be used; in flood work, a coefficient of from 90 to 95%; and for streams at ordinary stages, from 85 to 90 per cent.

The results of a large number of vertical velocity-curve measurements, giving a comparison of the two-tenths-eight-tenths method, the six-tenths method, and the sub-surface method are given in Table 4.

Independent discharge measurements, as a rule, are of but little value in hydraulic work unless they are taken at stages which are known to be either extremely low or extremely high. In ordinary work it is necessary to make a series of measurements which, with daily gauge heights of the flow of the stream, enable the computation of the total flow of the stream and also its distribution. In connection with the individual measurements, therefore, it is necessary to observe gauge heights and take full notes of the conditions under which the measurement is made, in order to enable the construction of a station rating curve and estimate the daily discharge.

Computations of Current-Meter Discharge Measurements.

The computations of current-meter discharge measurements are usually made to determine: (*a*) the total area, (*b*) the mean velocity, and (*c*) the total discharge of the stream. The observed data from which these computations are made consist of (*a*) soundings at known intervals across the stream, (*b*) the mean velocity in the vertical at each sounding point, and (*c*) the distance between the points of measurement (Table 5).

The mean velocity, area, and discharge are computed independently

for each partial area included between perpendiculars from successive measuring points. The total discharge is the sum of the partial discharges. The computation of the partial discharge eliminates errors which would arise from the distribution of conditions existing in one part of the channel to parts to which they do not apply. The mean velocity is determined by dividing the total discharge by the total area of the cross-section.

The formulas used in connection with computations, may, in general, be classed as rectilinear and curvilinear—depending on the assumption that the bed of the stream and the horizontal velocity-curve vary from the measuring point either as a straight line or as a series of curved lines. A comparison of the computation of discharge measurements by various formulas, has been prepared by J. C. Stevens, Assoc. M. Am. Soc. C. E., District Engineer of the United States Geological Survey.*

In this discussion it is shown that the rectilinear formula:

$$D = l_1\left(\frac{d_0 + d_1}{2}\right)\left(\frac{v_0 + v_1}{2}\right) + l_2\left(\frac{d_1 + d_2}{2}\right)\left(\frac{v_1 + v_2}{2}\right) + \ldots\ldots\ldots\ldots\ldots\ldots l_n\left(\frac{d_{n-1} + d_n}{2}\right)\left(\frac{v_{n-1} + v_n}{2}\right)$$

gives the most accurate results and is readily used.

In this formula, d_0, d_1, d_2.....d_n and v_0, v_1, v_2.....v_n are the depths and velocities at the respective measuring points, a_0, a_1, a_2.....a_n, which are spaced at the distances, l_1, l_2, l_3.....l_n (Fig. 7). The area between any two perpendiculars at measuring points is equal to the mean of the depths at such points multiplied by the distance. Similarly, the mean velocities for the area between the two perpendiculars is equal to the mean of the velocities at the two perpendiculars. The product of this area and the mean velocity is equal to the discharge for the partial area between the two perpendiculars. Summing these partial areas and discharges gives the total area and total discharge. The mean velocity for the section is equal to the total discharge divided by the total area. Table 5 gives the field notes and computations for a typical current-meter measurement.

Low-Water Measurements.

At many stations the velocity of the river at low stages is so small that it may be advisable to find a section near by in which

Engineering News, June 25th, 1908.

MEASUREMENTS PRIOR TO 1905.												
Susquehanna	McCall Ferry, Pa	73	3.3-30.0	1.21-5.80	68		0.94		0.92	1.07		
Susquehanna	McCall Ferry, Pa	68	5.0-36.0	1.40-9.70	73		0.97		0.90			
Susquehanna	Harrisburg, Pa	20	3.2- 8.0	1.52-2.71	61		1.01		0.85	1.08		
Susquehanna	Binghamton, N. Y	36	2.5- 8.1	0.80-4.86	61		0.99			0.96		
Susquehanna	Binghamton, N. Y	34	1.7- 7.4	0.46-3.38	62		0.98			1.04		
Chenango	South Cairo, N. Y	47	2.1- 7.0	0.27-3.91	58		1.02		0.83			
Catskill Creek	South Cairo, N. Y	6	2.1- 4.8	0.75-4.12	59		1.02		0.82			
Delaware (E. Br.)	Hancock, N. Y	12	1.7- 4.2	1.67-4.45	58		1.02		0.83			
Delaware (W. Br.)	Hancock, N. Y	53	1.9-14.0	0.65-5.04	62		1.00		0.88			
Esopus Creek	Kingston, N. Y	68	0.9- 5.2	0.42-7.62	58		1.01		0.69			
Esopus Creek	Olivebridge, N. Y	22	2.1- 5.0	0.26-1.75	59		1.04		0.79			
Fishkill Creek	Glenham, N. Y	36	2.1- 6.2	0.67-3.66	58		1.01		0.83			
Housatonic	Gaylordsville, Conn	21	4.4- 8.2	0.35-2.55	61		0.99		0.84			
Rondout Creek	Rosendale, N. Y	11	1.1- 6.6	0.52-3.20	58		1.01		0.80			
Schoharie Creek	Prattsville, N. Y	33	1.8- 7.3	0.24-2.15	61		0.99		0.88			
Tenmile	Dover Plains, N. Y	52	3.3-17.3	0.18-4.72	64		0.98		0.85			
Wallkill	New Paltz, N. Y	17	2.5- 4.7	0.58-2.11	58		1.03		0.78			
Wappinger Creek	Wappinger Falls, N. Y	8	2.5- 4.9	0.92-2.30	62		1.00		0.81	1.11		
Appomattox	Mattoax, Va	6	3.0- 4.4	1.88-2.37	60		1.01		0.88	1.08		
James	Cartersville, Va	8	1.8- 3.3	0.87-1.96	61		0.99		0.81	1.01		
Roanoke	Roanoke, Va	20	2.0- 6.0	1.35-3.11	66		0.96		0.95	1.12		
Staunton	Randolph, Va	42	1.3- 3.8	0.73-3.00	62		0.99		0.86	1.01		
Dan	Madison, N. C	13	3.5- 9.0	0.90-2.41	64		0.98		0.90	1.09		
Dan	South Boston, Va	3	1.7- 2.5	1.62-2.01	58		1.02		0.83	0.97		
Reddie	North Wilkesboro, N. C	33	2.7- 6.4	1.20-3.16	66		0.96		0.91	1.12		
Yadkin	North Wilkesboro, N. C	10	3.1-11.3	0.96-3.23	59		1.02		0.81	1.06		
Yadkin	Salisbury, N. C	39	1.9- 5.0	0.74-2.84	61		1.00		0.82	1.04		
Catawba	Morgantown, N. C	2	6.3- 7.0	0.49-0.74	58		1.00		0.88	1.08		
Catawba	Mount Holly, N. C	1	3.5	1.54	59		1.01		0.80	1.05		
Catawba	Belmont, N. C	4	12.3-17.7	1.33-1.83	58		1.01		0.90	1.13		
Wateree	Camden, S. C	4	5.0- 8.9	2.79-4.00	65		0.96		0.89	1.22		
Broad	Alsten, S. C	4	5.0- 8.9	2.79-4.00	65		0.96		0.89	1.22		
Saluda	Waterloo, S. C	3	3.0- 8.5	1.76-2.01	62		0.97		0.82	1.03		
Watauga	Elizabethton, Tenn	10	2.6- 5.3	0.34-1.01	61		0.98		0.82	1.07		
South Fork Holston	Bluff City, Tenn	13	1.9- 4.1	1.08-6.77	62		1.02		0.80	1.05		
Nolichucky	Greeneville, Tenn	22	1.6- 5.1	0.49-2.69	59		1.02		0.80	1.02		
French Broad	Oldtown, Tenn	9	4.2- 8.0	0.53-2.18	62		0.99		0.86	1.07		
Pigeon	Newport, Tenn	4	4.4-11.6	0.82-1.14	61		0.99		0.91	1.15		
Tuckaseegee	Bryson City, N. C	36	3.5- 8.5	0.50-3.39	64		0.97		0.93	1.09		
Little Tennessee	Judson, N. C	11	2.7- 6.0	3.84-7.45	59		1.03		0.83	1.06		
Mean of 910 curves					61		1.00		0.85	1.07		
Highest			36.0	9.70	73		1.04		0.95	1.22		
Lowest			1.1	0.18	58		0.94		0.78	0.96		

* From "River Discharge."

TABLE 4.—SUMMARY OF VERTICAL VELOCITY CURVES IN OPEN CHANNELS.*

Stream and locality.		Number of curves.	Range of depth, in feet.	Range of velocities, in feet per second.	Depth of thread of mean velocity, in percentage of total depth.	Depth of thread of maximum velocity, in percentage of total depth.	Coefficients for Reducing to Mean Velocity.					
							Six-tenths depth.	Mid-depth.	Top.	Mean of top and bottom.	$\frac{0.2+0.8}{2}$	$\frac{T+2M+B}{4}$
MEASUREMENTS SINCE 1905.												
Ohio	Wheeling, W. Va.	14	5.3-20.8	0.92-4.43	66	13	0.98	.....	0.93	.....	1.005	.....
Monongahela	Lock No. 4, Pa.	7	10.9-17.5	0.74-1.13	68	0	0.96	.....	0.85	.....	0.990	.....
St. Mary	Cardston, Alberta.	8	2.7- 3.9	3.78-6.32	58	.....	1.01	.....	0.88	.....	1.001	.....
Jefferson	Sappington, Mont..	7	2.7- 5.7	1.78-3.45	63	.....	0.97	.....	0.84	.....	0.997	.....
Gallatin	Logan, Mont.	7	3.7- 5.5	0.88-3.85	62	.....	0.98	.....	0.85	.....	0.991	.....
Missouri	Cascade, Mont.	22	2.1-12.0	0.79-3.04	62	.....	0.99	.....	0.88	.....	1.002	.....
Marias	Shelby, Mont.	20	1.9- 4.2	0.57-4.52	60	.....	1.01	.....	0.86	.....	1.004	.....
Yellowstone	Glendive, Mont.	11	4.3- 8.5	2.42-3.79	60	.....	1.00	.....	0.89	.....	1.003	.....
Bighorn	Fort Custer, Mont.	8	3.9- 7.4	1.22-2.60	59	.....	1.00	.....	0.86	.....	0.998	.....
Shoshone	Cody, Wyo.	16	4.3- 7.3	4.93-8.45	58	13	1.02	0.96	0.87	.....	1.00	.....
North Platte	Saratoga, Wyo	8	3.2- 5.1	1.82-3.61	63	26	0.98	.....	0.94	1.31	1.026	1.09
Pecos	Dayton, N. Mex.	10	4.1- 8.2	1.68-2.81	58	.....	1.03	.....	0.82	.....	1.003	.....
Pecos	Pecos, Tex.	6	2.2- 6.0	1.52-3.06	63	.....	0.97	.....	0.98	.....	1.004	.....
Green	Greenriver, Wyo.	36	2.2- 6.4	1.05-4.12	58	14	1.01	.....	0.90	1.20	1.026	1.06
Yampa	Steamboat Springs, Colo.	8	4.2- 5.2	7.13-8.42	59	7	1.01	.....	0.83	1.10	1.003	1.02
Grand	Hot Sulphur Springs, Colo.	6	2.8- 8.2	0.89-2.87	63	23	0.98	.....	0.85	1.30	1.01	1.03
Grand	Kremmling, Colo.	15	9.2-27.5	1.11-4.02	65	24	0.97	.....	0.93	1.16	1.001	1.03
Gunnison	Cimarron, Colo.	9	4.3-15.5	3.83-8.70	64	19	0.99	.....	0.92	1.06	1.003	1.02
Gunnison	Cory, Colo.	5	4.9-10.4	1.63-9.59	63	17	0.99	.....	0.92	1.09	1.007	1.01
Big Blackfoot	Bonner, Mont.	7	4.2- 6.8	1.41-2.49	62	.....	0.98	.....	0.85	.....	1.016	.....
Clark Fork	Missoula, Mont.	5	5.4- 7.6	1.72-2.41	60	10	1.00	.....	0.79	.....	0.997	.....
Yakima	North Yakima, Wash.	7	4.9- 7.6	1.58-2.56	59	.....	1.01	.....	0.83	1.06	0.999	1.00
Yakima	Yakima, Wash.	50	3.2-12.6	0.65-8.56	59	.....	1.01	.....	0.85	1.04	1.003	0.99
Yakima	Kiona, Wash.	20	2.0- 6.1	0.25-3.24	61	.....	0.98	.....	0.84	1.05	1.00	0.98
Kachess	Easton, Wash.	5	3.4- 4.3	1.44-4.78	61	.....	0.99	.....	0.85	1.04	1.009	0.99
Clealum	Roslyn, Wash.	20	4.7-11.7	0.37-3.89	58	.....	1.01	.....	0.84	1.09	0.995	1.02
Naches	North Yakima, Wash.	25	2.4- 6.0	0.65-7.38	58	.....	1.01	.....	0.85	1.04	0.998	0.99
Snake	Minidoka, Idaho.	14	4.0-10.7	1.13-3.05	62	20	0.99	.....	0.89	.....	0.994	.....
Big Lost	Mackay, Idaho.	7	2.4- 3.7	1.23-3.85	71	42	0.95	.....	0.95	.....	0.993	.....
Palouse	Hooper, Wash.	28	1.6- 8.0	0.97-6.49	60	.....	1.00	.....	0.84	1.06	1.004	1.00
John Day	McDonald, Ore.	15	3.6- 5.9	1.87-4.44	65	.....	0.98	.....	0.89	.....	.....	.....
Umatilla	Pendleton, Ore.	13	2.2- 7.5	0.47-2.73	64	.....	0.97	0.90	0.87	.....	0.970	.....
Deschutes	Lava, Ore.	10	3.0- 4.1	0.88-1.98	66	.....	0.99	0.94	0.92	.....	0.994	.....
West Fork Deschutes	Lava, Ore.	27	1.7- 7.1	0.58-2.65	64	.....	0.98	0.96	0.89	.....	0.992	.....
Mean of 476 curves		.....	.....	.....	62	.....	0.99	.....	0.87	.....	1.001	.....
Highest		.....	27.5	9.59	71	.....	1.03	.....	0.98	.....	1.026	.....
Lowest		.....	1.6	0.25	58	.....	0.95	.....	0.79	.....	0.970	.....

TABLE 4.—*Continued.*

Stream and locality.	Number of curves.	Range of depth, in feet.	Range of velocities, in feet per second.	Depth of thread of mean velocity, in percentage of total depth.	Depth of thread of maximum velocity, in percentage of total depth.	Coefficients for Reducing to Mean Velocity.					
						Six-tenths depth.	Mid-Depth.	Top.	Mean of top and bottom.	$\frac{0.2+0.8}{2}$	$\frac{T+2M+B}{4}$
Shallow Streams with Rough Beds.											
St. Mary.....Dam Site, Mont.....	24	1.8- 4.2	2.69-6.36	60	9	1.00	0.93	0.86	.....	1.01	.....
Swiftcurrent Creek....Babb, Mont.....	16	1.6- 3.2	3.28-6.81	57	0	1.03	0.96	0.87	.....	1.004	.....
South Fork Shoshone.Marquette, Wyo.....	20	1.8- 3.6	3.61-6.79	58	0	1.02	0.93	0.82	.....	1.016	.....
Ana.....Summer Lake, Ore.....	6	1.8- 2.4	1.83-3.37	63	10	0.97	0.92	0.89	.....	1.00	.....
Tieton.....North Yakima, Wash.....	14	1.8- 4.5	4.42-8.04	56	0	1.04	0.96	0.80	.....	1.00	0.98
Wallowa.....Wallowa, Ore.....	4	2.2- 2.9	4.16-4.93	58	0	1.04	0.94	0.78	.....	1.004	.....
Kruzgamepa.....Salmon Lake, Alaska.....	108	1.6- 3.6	1.43-4.70	64	23	0.97	0.91	0.86	.....	1.004	0.99
Small streams.....Seward Peninsula, Alaska...	27	1.7- 2.5	2.15-3.47	61	14	1.00	0.93	0.84	....	1.004	1.00
Mean of 219 curves.....	.....	.....	.....	60	7	1.01	0.92	0.84	.....	1.005	0.99
Highest.....	.....	4.5	8.04	64	23	1.04	0.96	0.89	.....	1.016	.....
Lowest.....	.....	1.6	1.43	56	0	0.97	0.91	0.78	.....	1.00	.....

TABLE 5.—Current-Meter Discharge Measurement Notes.

Date, Dec. 12, 1908, 9.15 A. M. Stream, Rock Cr. Party, F. F. Henshaw. Locality, Mass. Ave. Bridge, D. C. Meter No. 112. Gauge height, beg. 1.97, end 2.03, mean 2.00. Total area, 67. Mean velocity, 2.15. Discharge, 144.

Observations.					Computations.						
					Velocity.						
Distance from initial point.	Depth.	Depth of observation	Time, in seconds.	Revolutions.	At point.	Mean in vertical.	Mean in section.	Mean depth.	Width.	Area.	Discharge.
0	0.0	.....	.....	0	.....	0.00	.....	.....	.....	.....	.....
							0.48	0.2	2	0.4	0.2
2	0.4	0.21	46	20	.....	0.96					
							1.22	0.5	2	1.0	1.2
4	0.6	0.36	44.4	30	.....	1.49					
							1.60	0.75	2	1.5	2.4
6	0.9	0.54	38.6	30	.....	1.71					
		0.23	42.4	50	2.63		1.88	1.0	4	4.0	7.5
10	1.1	0.92	44	30	1.50	2.06					
		0.3	31.4	40	2.84		2.16	1.3	2	2.6	5.6
12	1.5	1.2	39.2	30	1.68	2.26					
		0.36	39.8	50	2.79		2.24	1.65	2	3.3	7.4
14	1.8	1.44	39.6	30	1.67	2.23					
16	1.9	.....	.....	.....	.....	.....	2.28	1.9	4	7.6	17.3
		0.38	38.8	50	2.87						
18	1.9	1.52	36.8	30	1.80	2.34					
		0.4	36.6	50	3.05		2.42	1.95	4	7.8	18.9
22	2.0	1.6	34.2	30	1.94	2.50					
		0.38	38	50	2.93		2.49	1.95	4	7.8	19.4
26	1.9	1.52	43.8	40	2.02	2.48					
		0.36	39		2.85		2.57	1.85	4	7.4	19.0
30	1.8	1.44	45	50	2.47	2.66					
		0.32	45	50	2.47		2.35	1.7	4	6.8	16.0
34	1.6	1.28	40.8	30	1.62	2.04					
		0.26	32		2.08		1.95	1.45	4	5.8	11.3
38	1.3	1.04	40.4	30	1.63	1.86					
		0.24	49.4	50	2.25		1.90	1.25	2	2.5	4.8
40	1.2	0.96	40	30	1.65	1.95					
							1.88	1.15	2	2.3	4.3
42	1.1	0.66	36.4	30	.....	1.82					
							1.65	1.0	3	3.0	5.0
45	0.9	0.57	44.8	30	.....	1.48					
							1.36	0.9	3	2.7	3.7
48	0.9	0.54	35.2	20	.....	1.25					
							0.88	0.45	1	0.4	0.4
49	0.0	.....	.....	.....	.....	0.50					
									49	66.9	144.4

the meter measurement may be made by wading. In order to obtain a suitable section, it may be necessary to cut off part of the flow by damming the stream and modifying the channel so as to get sufficient depth and velocity for measuring.

For such measurements, the meter clamped on the rod (Plate XXIII) is the most satisfactory; in this the Price acoustic attachment is especially useful. In making a wading measurement, a tape line is

stretched across the stream in order to mark the measuring points. The engineer stands below this tape line and to one side of the meter (Fig. 1, Plate XXVII) in order that he may not disturb the action of the meter. Iron rods, $\frac{3}{8}$ in. in diameter and 3 or 4 ft. long, and having a split at the top, may be used conveniently to support the tape when making such measurements.

Measurements Under Ice.

Measurements under ice should always be made by the vertical velocity-curve method, or by the two-tenths-eight-tenths method, the latter, by many comparisons with vertical velocity-curves, having been shown to give results closely approximating the mean velocity. In work of this class, if the water is not too deep and the velocity too great, the meter may best be held on the rod.

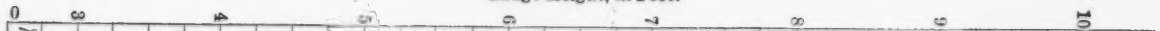
Measurements in Artificial Channels.

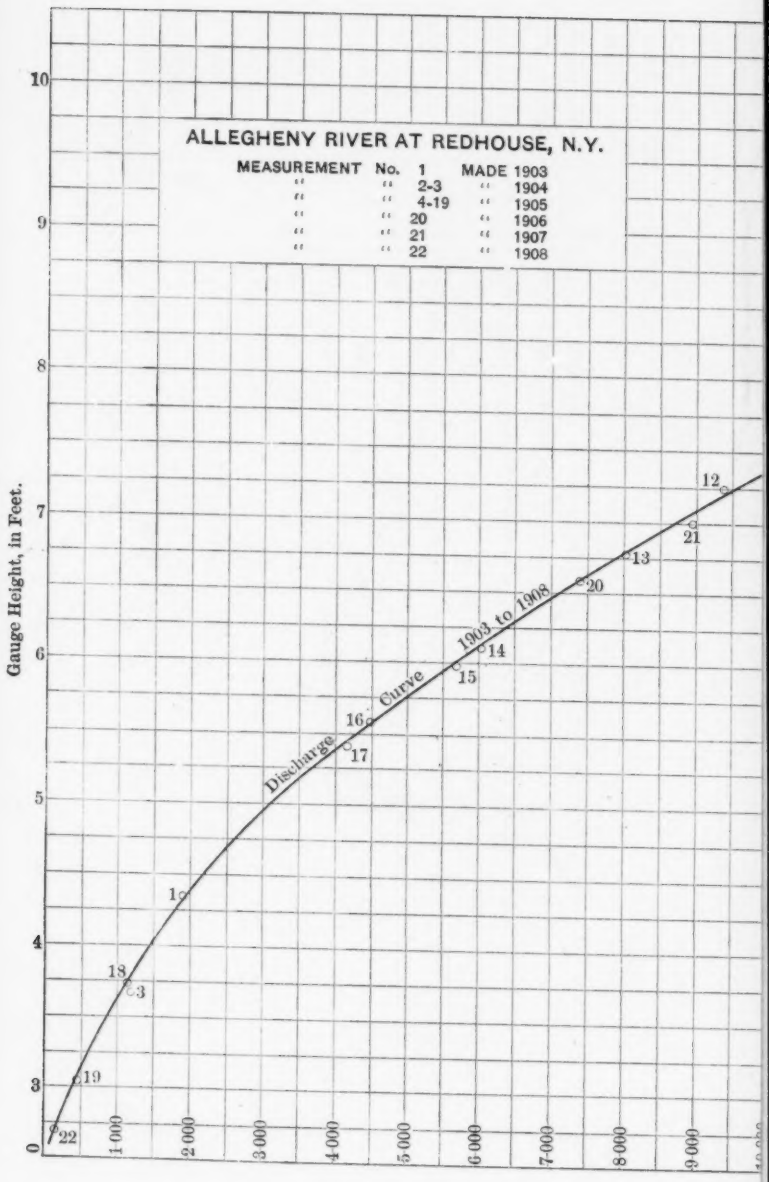
Current-meter measurement of the flow of water in artificial channels must be made with special care, as the laws of flow for open channels are not always applicable to artificial ones, because the water level may be subjected to disturbing influences such as undercurrents caused by intakes and outlets at rapid velocities.

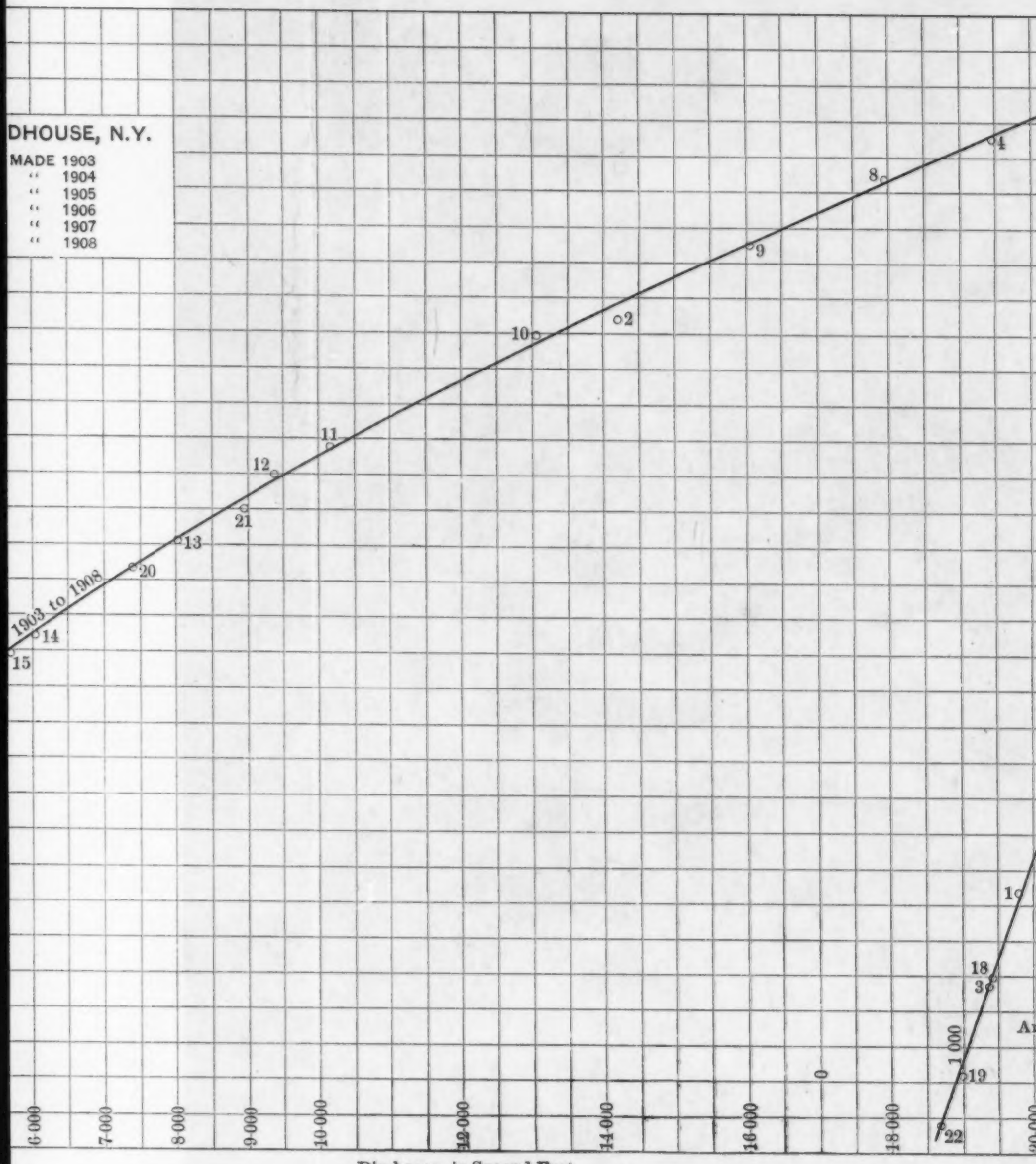
Accuracy and Reliability of the Current Meter.

No attempt will be made herein to review the numerous discussions on the accuracy and use of the current meter. It is unfortunate that, in these discussions, most of the adverse arguments have been based on personal opinions rather than on the results of extended meter observations, and that from them there has resulted somewhat of a prejudice against the meter by those who are not familiar with its use and limitations. This prejudice, however, is rapidly disappearing, and, with increased use, the meter is coming to be recognized as a standard instrument in hydraulic investigations.

In the consideration of the accuracy of the results obtainable with the current meter, account should be taken of the use to be made of the data. It must be remembered that both the total flow of the stream and its distribution over the drainage area are constantly changing, and that the conditions existing at any time may probably never occur again. The flow which may be expected in any given stream, therefore, can be determined only by studying a series of

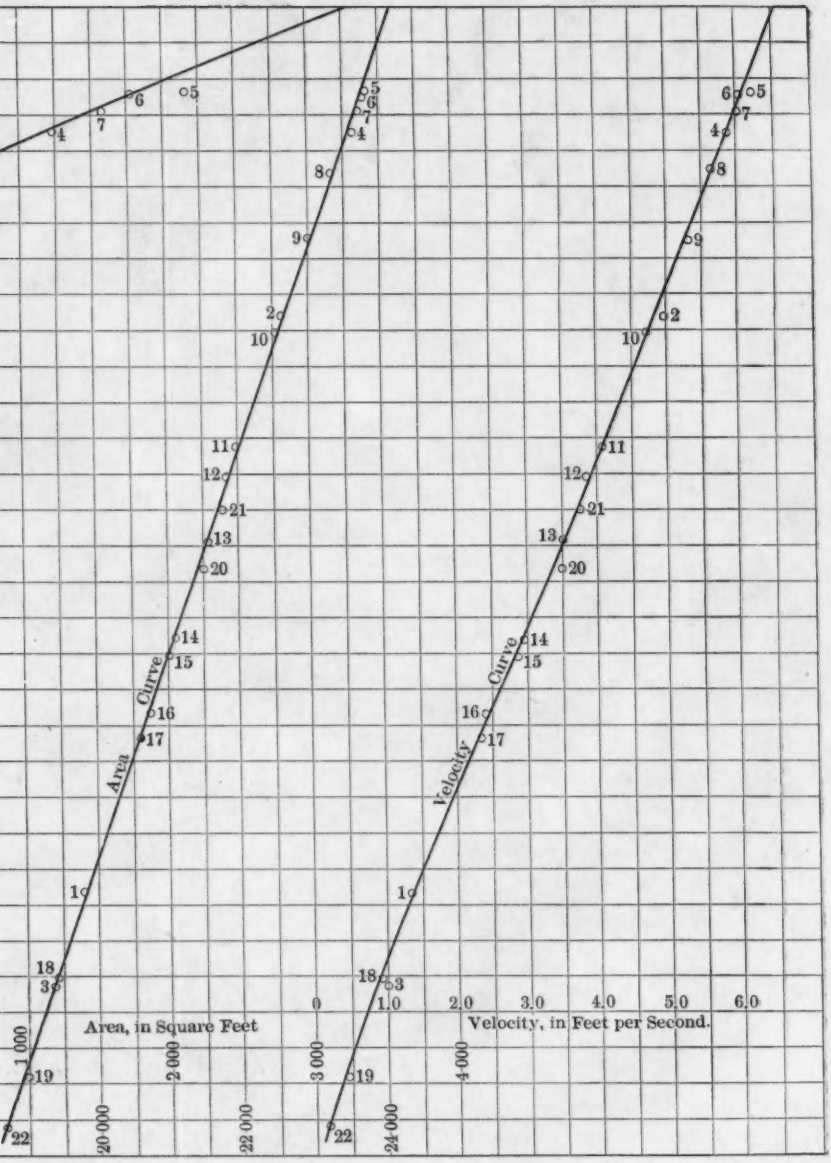

Gauge Height, in Feet.
0
3
4
5
6
7
8
9
10


ALLEGHENY RIVER AT REDHOUSE, N.Y.
MEASUREMENT No. 1 MADE 1903
" " 2-3 " 1904
" " 4-19 " 1905
" " 20 " 1906
" " 21 " 1907
" " 22 " 1908
Discharge Curve 1903 to 1908
Gauge Height, in Feet.
0
3
4
5
6
7
8
9
10
1 000
2 000
3 000
4 000
5 000
6 000
7 000
8 000
9 000



Discharge, in Second-Feet.

PLATE XXIX.
PAPERS, AM. SOC. C. E.
AUGUST, 1909.
HOYT ON
THE USE AND CARE OF THE CURRENT METER.



records extending over a long period; for this reason ultra-refined measurements would have no more practical value than records having a reasonable degree of accuracy.

As with most instruments, the accuracy and reliability of the current meter depend largely on the care taken in the measurement and the method used. In general, the errors which have been attributed to the meter have been due wholly to the fact that it was operated by men who were unfamiliar with it and with the conditions under which it could be used.

Its reliability is best shown by the results of the measurements taken at gauging stations on streams with permanent beds. These results show that measurements taken with different meters and by different operators at intervals throughout a series of years plot on a well-defined curve, Plate XXIX.

An examination of the rating tables (Table 3) for various meters indicates that, barring a very slight variation due to slight differences in manufacture, a standard table could be used for all meters and still bring the results within the limit of error which should be expected in this work. The re-ratings of the meters, as used in the field, also indicate that they are not subject to great change, if they have not been damaged and if the pivots are kept in good condition. It is also noted that the meter is most reliable for velocities greater than 1 ft. per sec., which is the most common velocity in stream gauging.

A review of all the available information in regard to the accuracy of the meter, and of the data collected by the engineers of the United States Geological Survey during the last two decades, shows that the current meter is wholly reliable for general stream-gauging work, and that, in general, the errors due to the instrument when properly used will not exceed 5 per cent. It is believed that the results of the greater number of measurements are in error less than that amount.

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AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

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COMPUTATION OF STRESSES IN OPEN-WEBBED ARCHES WITHOUT HINGES.*

By C. W. Hudson, M. Am. Soc. C. E.

The principles underlying the following method are often used in finding the stresses in certain classes of statically indeterminate structures built of elastic material, but, as far as the writer is aware, they have never been applied to finding the stresses in an open-webbed arch rib without hinges.

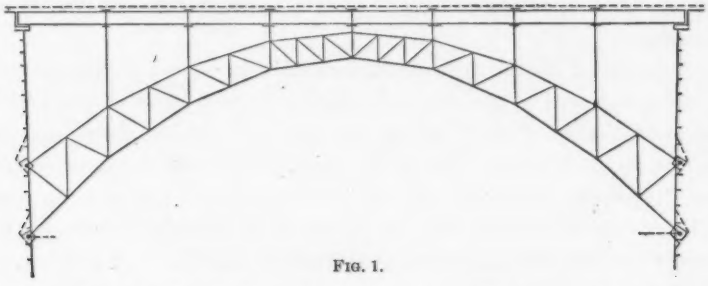

Fig. 1.

If such an arch rib be considered cut, at the crown, by a vertical plane, it is divided into two parts which are each statically determinate.

For the purpose of illustration, assume an arch rib as shown in Fig. 1. Consider the rib cut at the center by a vertical plane; this divides the rib into two parts as shown in Figs. 2 and 3, the vertical

* This paper will not be presented at any meeting, but written communications on the subject are invited for publication with it in *Transactions*.

member at the center being considered divided into two equal parts, one of which is taken with Fig. 2 and the other with Fig. 3.

It is perfectly clear that a load at any point on the left half produces stresses in the truss members of that half which may be determined by statics when each half acts as an independent cantilever truss, and that no stresses can be produced in the right half by this load when the two halves are not connected.

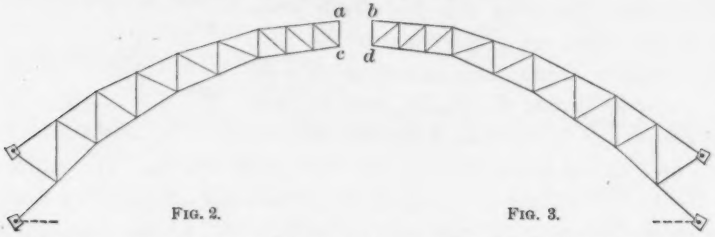

FIG. 2. FIG. 3.

If we suppose the two halves to be connected by joining the points, *a* and *b*, then, under any condition of loading, these two points have the same motion, as they are the same point. Therefore, for a load on the left half it is clear that the stress in any member of that half is that due to the given load when the half is considered as a separate structure modified by the stress acting between *a* and *b* caused by connecting these points, and the stress in any member of the right half is that due to the stress acting between *a* and *b* caused by connecting these points. No matter what the nature and direction of the force produced by joining *a* and *b*, it may be considered replaced by a horizontal and a vertical component.

If we suppose the two halves to be further connected by joining the points, *c* and *d*, then *a* and *b* have the same motion, and *c* and *d* have the same motion under any condition of loading. Therefore, for a load on the left half and the right and left halves connected at *a* and *b* and at *c* and *d*, the stress in any member of the left half is that due to the load on that half, when the half acts as a simple cantilever modified by the forces produced at *a* and *c* by connecting *a* to *b* and *c* to *d*, and the stress produced in any member of the right half is that due to the forces at *b* and *d* caused by connecting *a* to *b* and *c* to *d*. It hardly need be mentioned that the forces, due to connecting *a* to *b* and *c* to *d*, at *a* and *c*, are equal and opposite to those at *b* and *d*, respectively.

No matter what the amount and direction of the forces acting between a and b, and c and d, they may each be replaced by their horizontal and vertical components. For the two halves connected and subjected to any loading, the forces produced by thus joining them may be represented by the four forces of Fig. 4, H_a, V_a, H_c, and V_c, the direction of which may be as shown or the opposite and the magnitude of which is to be determined. Knowing that for elastic structures deflections are proportional to the loads that produce them, four equations between the unknown forces, H_a, V_a, H_c, and V_c, and certain easily determined deflections for the half arch acting independently, may readily be written. The solution of the four simultaneous equations will determine the unknown forces acting at the crown of the arch for each half. These equations of condition for finding the unknown forces at the crown will now be written for a symmetrical arch. The general method is equally applicable to an unsymmetrical arch, however.

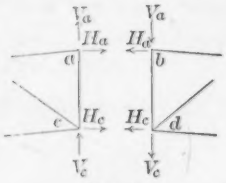

FIG. 4.

Let a load of 1 lb., at any point, x, cause the point, a, to take a new position, a', and the point, c, a new position at c', as shown in Fig. 5.

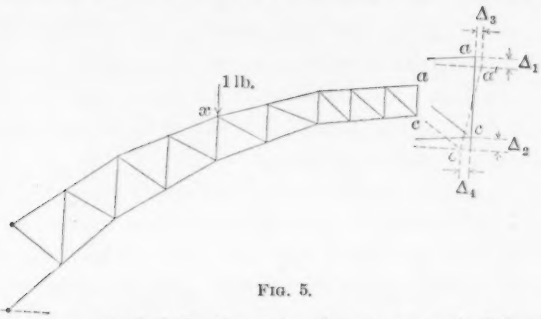

FIG. 5.

Then Δ_1 = vertical deflection of a due to a vertical load of unity at any point, x;

Δ_2 = vertical deflection of c due to a vertical load of unity at any point, x;

Δ_3 = horizontal deflection of a due to a vertical load of unity at any point, x;

Δ_4 = horizontal deflection of c due to a vertical load of unity at any point, x.

Let a vertical load of 1 lb. at a cause the point, a, to take a new position, a', and the point, c, a new position at c', as shown in Fig. 6.

Then d_1 = vertical deflection of a due to a vertical load of unity at a;

d_2 = vertical deflection of c due to a vertical load of unity at a;

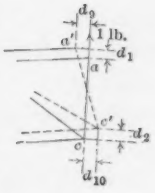

Fig. 6.

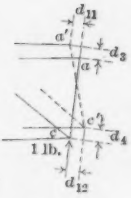

Fig. 7.

d_9 = horizontal deflection of a due to a vertical load of unity at a;

d_{10} = horizontal deflection of c due to a vertical load of unity at a.

Let a vertical load of 1 lb. at c cause the point, a, to take a new position, a', and the point, c, a new position at c', as shown in Fig. 7.

Then d_3 = vertical deflection of a due to a vertical load of unity at c;

d_4 = vertical deflection of c due to a vertical load of unity at c;

d_{11} = horizontal deflection of a due to a vertical load of unity at c;

d_{12} = horizontal deflection of c due to a vertical load of unity at c.

Let a horizontal load of unity at a, as shown in Fig. 8, cause the points, a and c, to take the new positions, a' and c', respectively.

Then d_5 = horizontal deflection of a due to a horizontal load of unity at a;

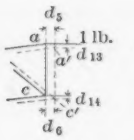

Fig. 8.

Fig. 9.

d_6 = horizontal deflection of c due to a horizontal load of unity at a;

d_{13} = vertical deflection of a due to a horizontal load of unity at a;

d_{14} = vertical deflection of c due to a horizontal load of unity at a.

Let a horizontal load of unity at c cause the points, a and c, to take the new positions, a' and c', respectively, as shown in Fig. 9.

Then d_7 = horizontal deflection of a due to a horizontal load of unity at c;

d_8 = horizontal deflection of c due to a horizontal load of unity at c;

d_{15} = vertical deflection of a due to a horizontal load of unity at c;

d_{16} = vertical deflection of c due to a horizontal load of unity at c.

The four simultaneous equations will now be written, from which the four unknown forces, V_a, V_c, H_a, and H_c, acting at the crown and due to a load of unity at any point, x, may be determined.

For the points, a and b, and the points, c and d, connected, the motion of a must equal the motion of b, and the motion of c must equal the motion of d.

The downward motion of $a = \Delta_1 - V_a d_1 - V_c d_3 + H_a d_{13} + H_c d_{15}$, and

the downward motion of $b = + V_a d_1 + V_c d_3 + H_a d_{13} + H_c d_{15}$, and these must be equal, as they are for the same point.

Therefore,

$$\Delta_1 - V_a d_1 - V_c d_3 + H_a d_{13} + H_c d_{15} = V_a d_1 + V_c d_3 + H_a d_{13} + H_c d_{15},$$

and

$$V_a d_1 + V_c d_3 = \frac{\Delta_1}{2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

The downward motion of $c = \Delta_2 - V_a d_2 - V_c d_4 + H_a d_{14} + H_c d_{16}$, and

the downward motion of $d = + V_a d_2 + V_c d_4 + H_a d_{14} + H_c d_{16}$, therefore,

$$\Delta_2 - V_a d_2 - V_c d_4 + H_a d_{14} + H_c d_{16} = V_a d_2 + V_c d_4 + H_a d_{14} + H_c d_{16},$$

and

$$V_a d_2 + V_c d_4 = \frac{\Delta_2}{2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

The motion to the right of $a = \Delta_3 - V_a d_9 - V_c d_{11} + H_a d_5 + H_c d_7$, and the motion to the right of $b = -V_a d_9 - V_c d_{11} - H_a d_5 - H_c d_7$, therefore,

$$\Delta_3 - V_a d_9 - V_c d_{11} + H_a d_5 + H_c d_7 = - V_a d_9 - V_c d_{11} - H_a d_5 - H_c d_7,$$

and

$$H_a d_5 + H_c d_7 = - \frac{\Delta_3}{2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

The motion to the right of $c = -\Delta_4 + V_a d_{10} + V_c d_{12} + H_a d_6 + H_c d_8$,

and the motion to the right of $d = +V_a d_{10} + V_c d_{12} - H_a d_6 - H_c d_8$, therefore,

$$-\Delta_4 + V_a d_{10} + V_c d_{12} + H_a d_6 + H_c d_8 = V_a d_{10} + V_c d_{12} - H_a d_6 - H_c d_8,$$

and

$$H_a d_6 + H_c d_8 = \frac{\Delta_4}{2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4)$$

Solving these four equations, the following values for the unknown forces are found, in terms of twelve easily determined deflections:

$$V_a = +\Delta_1 \frac{d_4}{2(d_1 d_4 - d_2 d_3)} - \Delta_2 \frac{d_3}{2(d_1 d_4 - d_2 d_3)}$$

$$V_c = -\Delta_1 \frac{d_2}{2(d_1 d_4 - d_2 d_3)} + \Delta_2 \frac{d_1}{2(d_1 d_4 - d_2 d_3)}$$

$$H_a = -\Delta_3 \frac{d_8}{2(d_5 d_8 - d_6 d_7)} - \Delta_4 \frac{d_7}{2(d_5 d_8 - d_6 d_7)}$$

$$H_c = +\Delta_3 \frac{d_6}{2(d_5 d_8 - d_6 d_7)} + \Delta_4 \frac{d_5}{2(d_5 d_8 - d_6 d_7)}$$

If the values of Δ_1 to Δ_4, and d_1 to d_8, as found by actual computation, are in the direction heretofore assumed, their numerical value is to be inserted in the foregoing expressions without regard to sign, if they are found to be in a direction opposite to that assumed, they must be inserted with a minus sign.

From Maxwell's Theorem, it is known that:

$$d_2 = d_3, \text{ and } d_6 = d_7.$$

For any particular configuration of members in the arch rib, very much simpler expressions for V_a and V_c may be written.

For the case assumed, to illustrate this paper, a load at any panel point other than a,

makes $\Delta_1 = \Delta_2$, and $d_3 = d_4$, and therefore $V_a = 0$.

A load at panel point, a,

makes $\Delta_1 = \Delta_2 + (d_1 - d_2)$, and d_3 is still $= d_4$, therefore $V_a = \frac{1}{2}$.

For any panel point of the rib loaded except a

$$V_c = \frac{\Delta_2}{2d_4}$$

And for panel point, a, loaded:

$$V_c = 0.$$

Restated, for any panel point loaded other than a

$$V_a = 0, \text{ and } V_c = \frac{\Delta_2}{2d_4}$$

And for panel point, a, loaded:

$$V_a = \frac{1}{2}, \text{ and } V_c = 0$$

For a configuration at the crown, as indicated in Fig. 10, that is, where the diagonals adjacent to the crown meet at the top of the crown vertical, $d_1 = d_2$, and d_3 and d_4 are no longer equal.

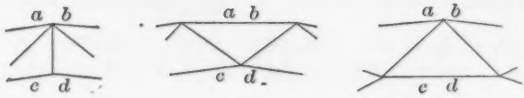

FIG. 10. FIG. 11. FIG. 12.

Then, for any panel point loaded other than c,

$\Delta_1 = \Delta_2$, and therefore,

$$V_a = \frac{\Delta_1}{2d_1}, \text{ and } V_c = 0$$

And for panel point c, loaded, $\Delta_1 = \Delta_2 - (d_4 - d_3)$, and $\Delta_2 = d_4$, and therefore,

$$V_a = 0, \text{ and } V_c = \frac{1}{2}$$

For such an arrangement of members at the crown as shown in Fig. 11, it is readily observed that

$$V_a = 0, \text{ and } V_c = \frac{\Delta_2}{2d_4}$$

for any panel point loaded.

For such an arrangement at the crown as shown in Fig. 12, it is seen that

$$V_a = \frac{\Delta_1}{2d_1}, \text{ and } V_c = 0$$

for any panel point loaded.

The expressions for the horizontal crown forces remain the same for all the cases selected.

Letting $2\ (d_5 d_8 - d_6^2) = n$, and remembering that $d_6 = d_7$, we have, for the value of the horizontal thrusts,

$$H_a = -\Delta_3 \frac{d_8}{n} - \Delta_4 \frac{d_6}{n}, \text{ and}$$

$$H_c = +\Delta_3 \frac{d_6}{n} + \Delta_4 \frac{d_5}{n}$$

The above expressions for the unknown crown forces have been written from assumed values of the twelve deflections of the half arch acting as a statically determined structure. For any actual problem, these deflections, of course, should be determined accurately, both as to direction and magnitude. The expressions for the value of the unknown crown forces will always be of the same general form as the above, but may differ in some of the signs of the terms.

The resulting plus values for V_a, V_c, H_a, H_c, indicate that they act as assumed, and the minus values that they act in opposite direction.

Temperature Stresses.—Let Fig. 13 show the motions of a and b, and c and d, for a rise in temperature when each half of the arch is considered as a separate structure, the full lines representing the normal figure and the dotted lines the figure changed by a rise in temperature.

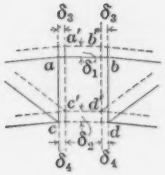

FIG. 13.

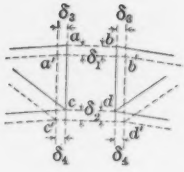

FIG. 14.

Let Fig. 14 show the motions of a and b, and c and d, for a corresponding fall in temperature under the same conception as before.

Then δ_1 = vertical deflection of a and b due to a certain change in temperature;

δ_2 = vertical deflection of c and d due to a certain change in temperature;

δ_3 = horizontal deflection of a and b due to a certain change in temperature;

and δ_4 = horizontal deflection of c and d due to a certain change in temperature.

Four equations for the unknown crown forces due to connecting the two halves may now be written for both a rise and a fall in temperature, using the notation as previously defined.

For a rise in temperature:

The upward motion of $a = +\ \delta_1 + V_a\, d_1 + V_c\, d_3 - H_a\, d_{13} - H_c\, d_{15}$, and the upward motion of $b = +\ \delta_1 - V_a\, d_1 - V_c\, d_3 - H_a\, d_{13} - H_c\, d_{15}$, for the two halves connected.

These motions must be equal, therefore, $V_a\, d_1 + V_c\, d_3 = 0$..... (5)

In an entirely similar manner, the three following equations may be written:

For a fall in temperature:

$$V_a d_2 + V_c d_4 = 0 \quad \ldots\ldots(6)$$
$$H_a d_5 + H_c d_7 = -\delta_3 \quad \ldots\ldots(7)$$
$$H_a d_6 + H_c d_8 = -\delta_4 \quad \ldots\ldots(8)$$
$$V_a d_1 + V_c d_3 = 0 \quad \ldots\ldots(5a)$$
$$V_a d_2 + V_c d_3 = 0 \quad \ldots\ldots(6a)$$
$$H_a d_5 + H_c d_7 = \delta_3 \quad \ldots\ldots(7a)$$
$$H_a d_6 + H_c d_8 = \delta_4 \quad \ldots\ldots(8a)$$

An examination of Equations (5), (6), (5*a*), and (6*a*), shows that V_a and V_c are zero, therefore, the only crown forces due to temperature are horizontal.

For a rise in temperature, remembering that $d_6 = d_7$, and making $(d_5 d_8 - d_6^2) = g$:

$$H_a = -\delta_3 \frac{d_8}{(d_5 d_8 - d_6^2)} + \delta_4 \frac{d_6}{(d_5 d_8 - d_6^2)} = -\delta_3 \frac{d_8}{g} + \delta_4 \frac{d_6}{g}, \text{ and}$$

$$H_c = -\delta_3 \frac{d_6}{(d_5 d_8 - d_6^2)} + \delta_4 \frac{d_5}{(d_5 d_8 - d_6^2)} = -\delta_3 \frac{d_6}{g} + \delta_4 \frac{d_5}{g}$$

For a fall in temperature,

$$H_a = +\delta_3 \frac{d_8}{(d_5 d_8 - d_6^2)} - \delta_4 \frac{d_6}{(d_5 d_8 - d_6^2)} = +\delta_3 \frac{d_8}{g} - \delta_4 \frac{d_6}{g}, \text{ and}$$

$$H_c = +\delta_3 \frac{d_6}{(d_5 d_8 - d_6^2)} - \delta_4 \frac{d_5}{(d_5 d_8 - d_6^2)} = \delta_3 \frac{d_6}{g} - \delta_4 \frac{d_5}{g}$$

The foregoing gives a simple method for finding the stresses in every member of the arch due to a load at any point, or for any desired change in temperature. Under this method, the work may be easily divided into several sets of independent and easily understood operations at the very beginning of the computations, thereby leaving only the small and easily managed theoretical part for the person in charge.

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PAPERS AND DISCUSSIONS

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WALNUT LANE BRIDGE, PHILADELPHIA

By George S. Webster and Henry H. Quimby, Members, Am. Soc. C. E.

To be Presented September 15th, 1909.

The bridge on the line of Walnut Lane, Philadelphia, over Wissahickon Creek, is notable for the great size of its main arch, for the novelty of some of its features, the character of its material, and the method of its construction. It is hoped that a description of it, with a discussion of the principles of its design, the reasons for its special features, and the lessons learned in the course of its construction will be of interest to the Profession.

The ravine of Wissahickon Creek, in Fairmount Park, is narrow and deep. It separates two populous sections of the City—Germantown and Roxborough—which are both on high ground, and heretofore most of the travel between them has been required to make a long detour, all the bridges being at a low level and the approaches to them too heavy in grade for business hauling.

By Ordinance of July 13th, 1905, City Councils authorized the construction of a high-level bridge on the line of Walnut Lane, the point where the ravine is narrowest and most advantageous for the crossing.

The required length of structure, of course, depended on the elevation of the deck, and as considerable grading of approaches was neces-

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sary—cutting for one and filling for the other—the rate of grade to be obtained on the approaches, together with the estimated money value of each percent. of reduced street gradient, entered into the consideration of the problem of dimensions and design. The question was somewhat further involved by the fact that on the Germantown side the approach is from three streets. Fig. 1 exhibits the original profiles and the elevations and grades adopted. A portion of the filling for the west approach was obtained from the cut of the east approach, and was carted across the bridge.

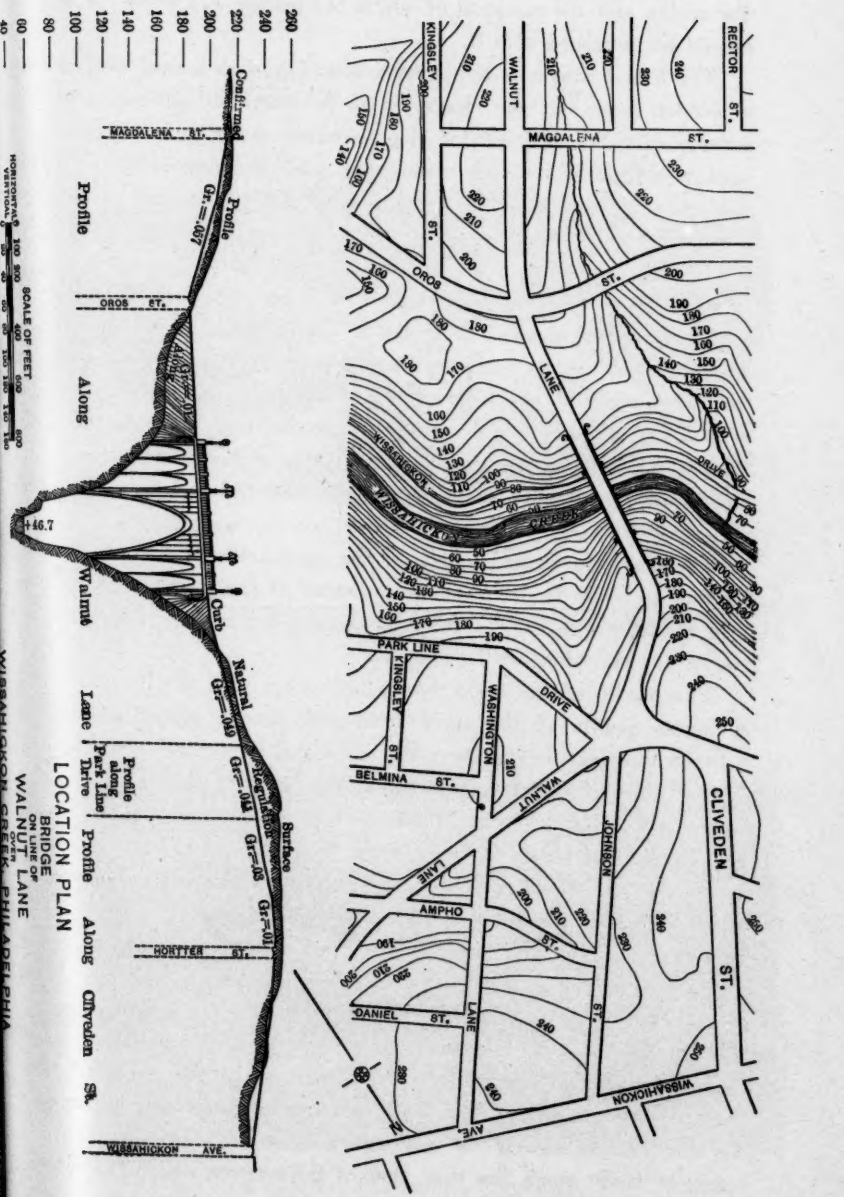
The bridge deck was given an ascending grade to the east—1½% to the middle of the main span and 1% from there to the east abutment. This change of grade was made in order to produce a slight crowning and relieve the elevation, or side view of the bridge, of the optical illusion of a dished appearance in the deck over the arch. This crowning is equivalent to a local grade of 1 in 400 on each side.

The grades adopted give the floor at the middle an elevation of 147 ft. above ordinary water level in the creek. As the depth of the structure at the crown is 14 ft., the soffit is 133 ft. above the creek and about 103 ft. above the drive, giving an imposing and majestic sweep of arch that is very impressive.

As the two springing points are at the same elevation, the grade over the bridge, in making the spandrel piers on one side higher and therefore thicker, puts extra dead load on that side, and this is counterbalanced by making the walls of the spandrel piers on the other side 4½ in. thicker.

In considering the matter of type of structure and length of spans, it was regarded as very undesirable, for æsthetic reasons, that there should be any pier or column interposed between the creek and the pleasure drive which winds along the west bank at an elevation of about 30 ft. above the water. The situation clearly indicated an arch as the only suitable form of support for the roadway, and the necessity of clearing both the drive and the creek, while making reasonable provision for the future widening of the drive, taken in consideration with the profile of the rocky bluffs, fixed the location of the abutment piers and the span of the main arch, which was made 233 ft. between the vertical faces of the piers.

The extraordinary length of the main span gave to the structure a monumental character, and it became important that the remainder of


Profile Along Walnut Lane
Profile along Park Line Drive
Profile Along Cliveden St.
Profile
Gr.=.007
Gr.=.049
Gr.=.044
Gr.=.03
Gr.=.01
Crest
Curb
Natural
Regulation
Surface
+46.7
MAGDALENA ST.
OROS ST.
HORTTER ST.
WISSAHICKON AVE.
SCALE OF FEET
HORIZONTAL
VERTICAL
LOCATION PLAN
BRIDGE ON LINE OF WALNUT LANE OVER WISSAHICKON CREEK, PHILADELPHIA
KINGSLEY
WALNUT
MAGDALENA
RECTOR ST.
OROS
LANE
KISHACOQUILLAS
WISSAHICKON CREEK
PARK LINE
DRIVE
WASHINGTON
BELMINA ST.
JOHNSON
CLIVEDEN ST.
AMPHO
DANIEL ST.
WISSAHICKON AVE.

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the design and the material of which the bridge was to be constructed should be in accord with it.

The lowest first cost would have been found in a steel viaduct with a wooden floor, but the character of the surroundings—natural park scenery of rocky and wooded slopes—would cause such a structure to appear inappropriate and inadequate, and the possibility of future neglect of proper maintenance enhanced the importance of selecting a construction of the most permanent character.

The development of the sphere of usefulness of concrete, and the city's extended experience in the use of it for arches, walls, and floors, with new and improved methods of manipulation and construction, whereby more nearly uniform character and value are obtained, and the adequacy and permanence of the material are assured, together with the great economy obtained through its use, influenced the adoption of concrete as the structural material of the bridge throughout, and it was designed so as to require steel reinforcement at only a few points.

The most economical length of the approach spans was determined by trial to be 60 ft. from center to center of the piers, or 53 ft. clear. The conditions called for three of these spans on the west slope and two on the east slope of the ravine.

The profile of the rocky banks naturally fixed the location of the springing points of the main arch, and gave a graceful proportion between the clear height above the creek and the length of the span.

The type of structure selected is that of the stone arch bridge in Luxemburg, Europe, built in 1904, which is substantially two bridges side by side with the space between them floored over. The type is economical for such a structure, because large masonry arches of a width sufficient for their own independent stability have more than ample strength for any highway live load that may have to be carried, and any increase in width or strength would be utilized only in sustaining the weight of the added material. Thus, in the main span of this bridge, the arch ribs themselves constitute one-half of the whole load to be carried—both dead and live—and, as the live load is less than one-twelfth of the dead load, a single barrel of the full width of the floor would clearly be wasteful because its thickness could not safely be made much less than that of the narrow ribs. The width of the floor is 60 ft., while that of each of the two ribs is 18 ft.—giving

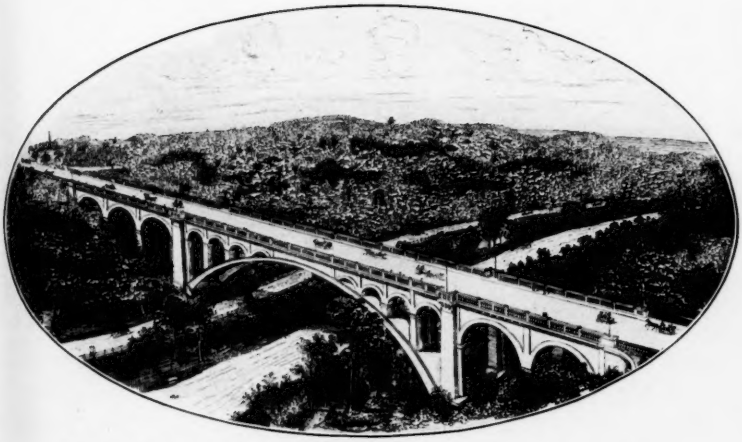

FIG. 1.—WALNUT LANE BRIDGE OVER WISSAHICKON CREEK, PHILADELPHIA.

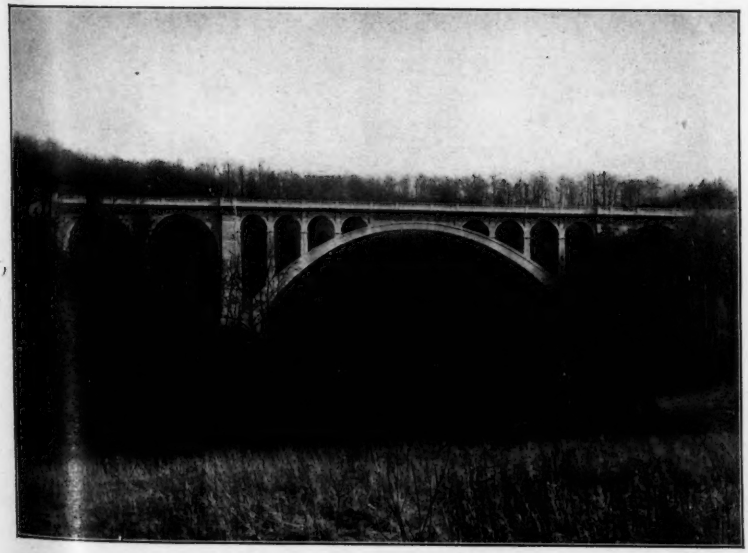

FIG. 2.—NORTH SIDE, WALNUT LANE BRIDGE.

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together 36 ft. of supporting arch, or only 60% of the floor width. The space between the ribs is 16 ft., leaving 4 ft. of floor to overhang on each side.

Each of the two halves or ribs of the bridge carries two vertical walls, 2 ft. 6 in. thick, which extend up to and carry the floor. These walls are set in 9 in. from the edges of the ribs, giving an actual overhang of 4 ft. 9 in. to the sidewalk floors on the outside of the bridge.

The three aisles thus formed, and spanned by the floor, are 20 ft. and 14 ft. for the middle and side aisles, respectively, from center to center of the walls.

These longitudinal walls are full spandrels over the 53-ft. arches, but on the main arch they are pierced by four 20-ft. arches over each haunch, leaving 60 ft. blank over the crown. These 20-ft. openings are flanged 6 in., in order to give the barrel or rib of each arch a width of 3 ft. 6 in., and pilasters of equal relief, 2 ft. 10 in. wide, were made between the arches to afford angles for concealing the vertical joints in the wall. In the 53-ft. arches the flanging of the walls is 9 in., making the ribs 4 ft. wide, but a sheeting of concrete, 18 in. thick, connects the ribs into pairs constituting the barrels, 18 ft. wide.

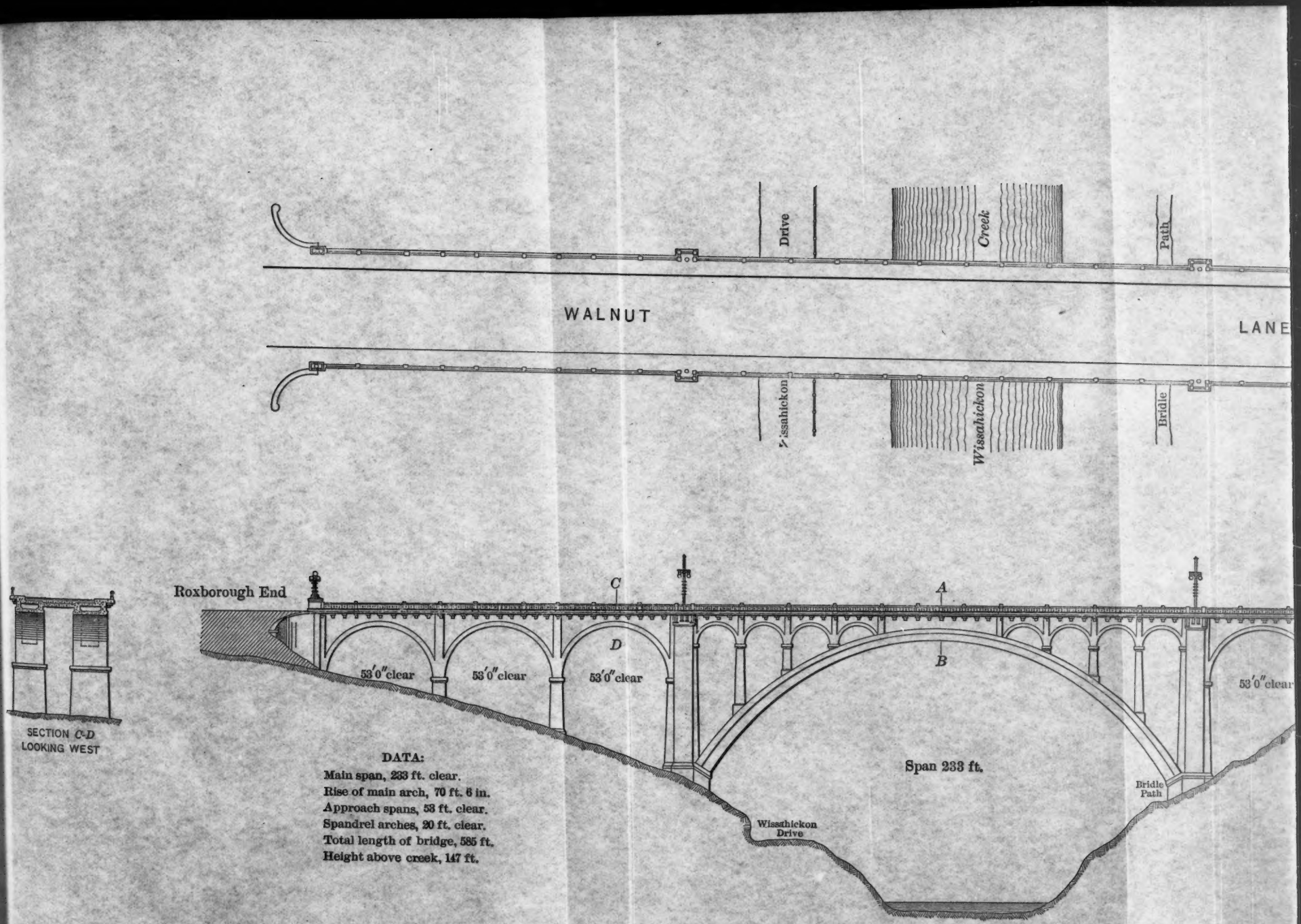
For architectural effect, the abutment piers at the ends of the main arch are widened to project 3 ft. beyond the general line of the structure, and they provide for refuge bays in the sidewalks of the deck. The twin shafts of these piers are hollow, 6 by 10 ft., giving a thickness of wall of 3 ft., which is increased below the water-tables by the pilasters under the arches. They have a floored chamber under the sidewalk, in order to give access to the electrical conduits and the bases of the light poles. The 10-in. vertical wrought-iron drain pipes from the grate-top inlets in the driveway gutters are secured to the inside wall of these shafts, and pass the drainage out through 16-in. cast-iron pipes built into the body of the pier and outletting on the rocky banks of the creek.

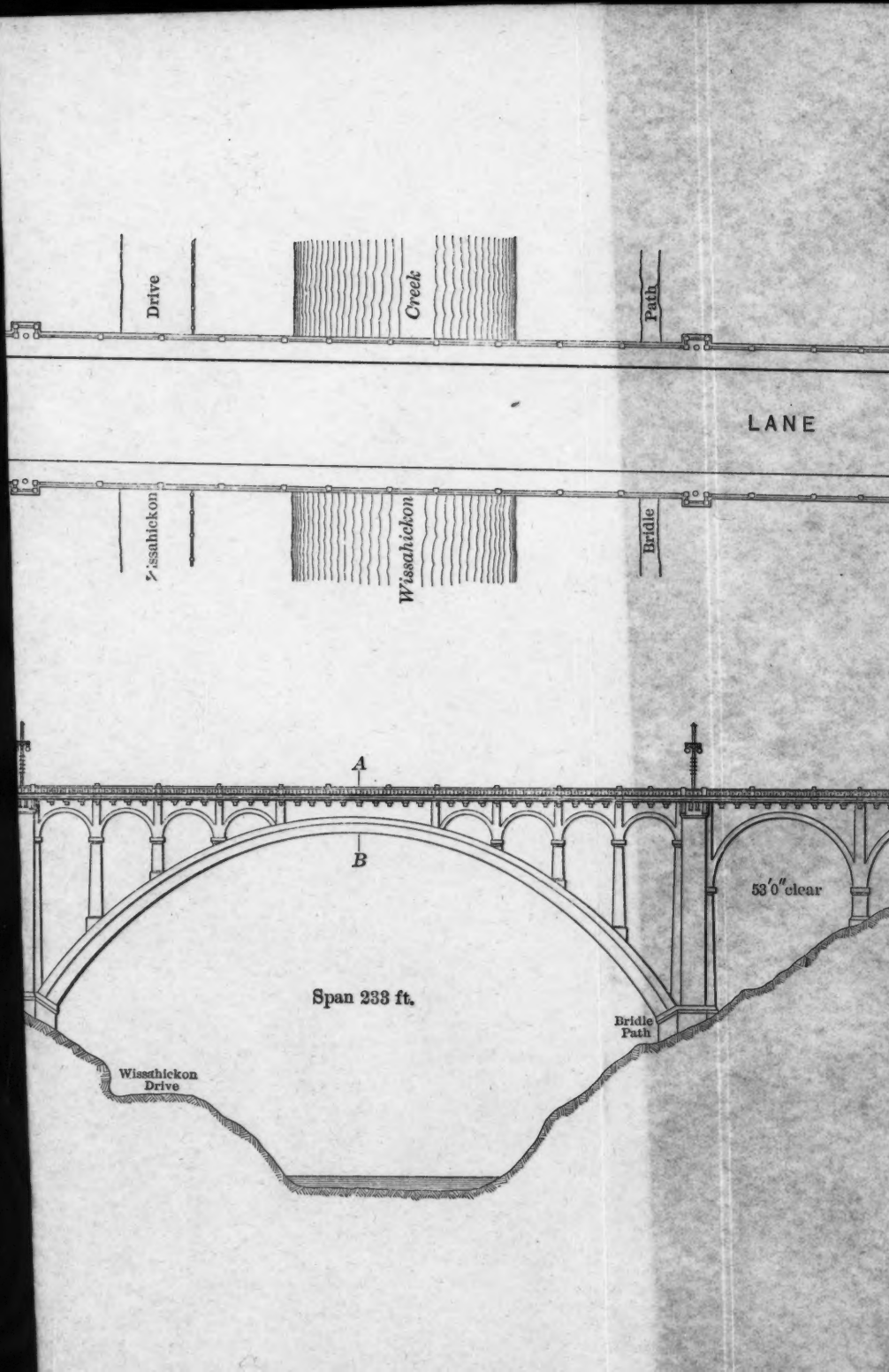
The floor structure consists of transverse rolled-steel I-beams, 6 ft. from center to center, with concrete jack-arches between them. The beams over the 20-ft. middle aisle are 20-in. 65-lb. per ft., and those over the 14-ft. side aisles are 15-in. 42-lb. per ft. The size of the floor members was determined by the assumed concentrated loads adopted as the standard for highway bridges in the City of Philadelphia, which consist of a truck, weighing 40 tons, on four wheels

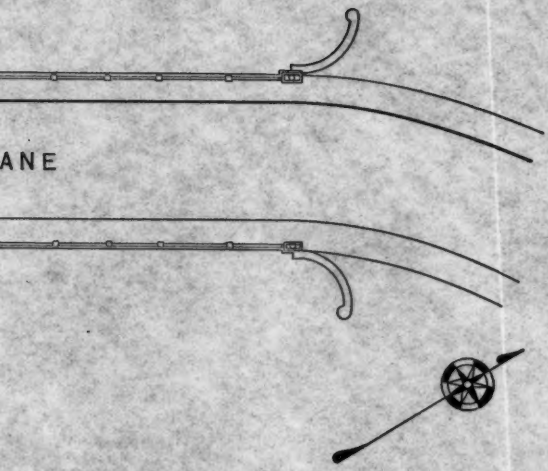
spaced 20 ft. between axles, and 6 ft. gauge, or two street railway cars side by side, weighing 20 tons on each truck.

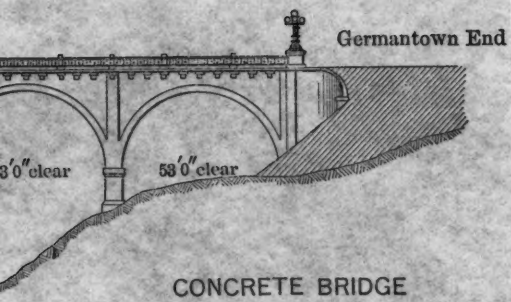
The concrete jack-arches can be depended on to distribute an axle load over two or more beams when the axles are more than 6 ft. apart, for a beam cannot deflect under its load without throwing some load on its immediate neighbors. The floor concrete is filled monolithically to 3 in. above the tops of the floor-beams. If the spring of the jack-arch be taken at the lower fillet of the beams, and a centroid of pressure be assumed 1½ in. above the tops of the beams, there is constituted for distributing purposes an arch of 12-ft. span, having a rise equal to the depth of the beam, and an effective thickness of 3 in. at the crown. A further assumption is made that the embedded beam will distribute a concentrated wheel load of 10 tons laterally over a space equal to the gauge of the wheels—6 ft. Each transverse foot of floor arch, therefore, is required to sustain 3 333 lb. of live load, and if one-half of this be distributed to the adjacent beams, the horizontal force developed in the distributing arch will give, in the worst case—that of the 15-in. side aisle beams—a unit stress of only 112 lb. per sq. in. Thus it appears unnecessary to compute the beams for more than one-half of an axle load when the axles are 12 ft. or more apart, but the standard weight of the beams used gives value equivalent to five-eighths of the 20-ton axle load at a fiber stress of 15 000 lb., without allowance for the reinforcing value of the encasing concrete. As it is found by experiment, confirming calculations based on conservative assumptions of adhesion of the concrete to the steel, that the embedment of rolled beams in concrete adds one-third to their carrying capacity, the margin possessed by the floor to carry heavier live loads than those specified is so great that the live load may be doubled without stressing the steel beyond common practice. The construction proves to be very rigid under rolling loads.

Careful estimates were made of the relative cost of floor construction of this type and of reinforced concrete—concrete beams and slabs reinforced with steel bars. The latter appeared to be the lower in cost by about $4.60 per panel, or less than $1 per lin. ft. of bridge, but at that time experience with construction of that class, for the severe service of heavy street traffic, was limited, and it was felt that the steel beams with independent strength were less of an experiment, and besides, the beams would furnish support for the forms of the floor


Drive
Creek
Path
WALNUT
LANE
Wissahickon
Wissahickon
Bridle
Roxborough End
C
A
D
B
53′0″clear
53′0″clear
53′0″clear
53′0″clear
SECTION C-D
LOOKING WEST
DATA:
Main span, 233 ft. clear.
Rise of main arch, 70 ft. 6 in.
Approach spans, 53 ft. clear.
Spandrel arches, 20 ft. clear.
Total length of bridge, 585 ft.
Height above creek, 147 ft.
Span 233 ft.
Wissahickon Drive
Bridle Path

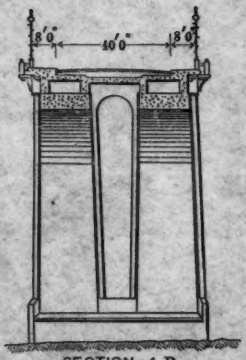

Drive
Creek
Path
LANE
Wissahickon
Wissahickon
Bridle
A
B
53′0″ clear
Span 233 ft.
Bridle Path
Wissahickon Drive





CONCRETE BRIDGE
ON LINE OF
WALNUT LANE
OVER THE
WISSAHICKON CREEK
PHILADELPHIA

Scale of Feet
20 0 20 40 60



SECTION *A-B*
LOOKING WEST

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arches.
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scaffold
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order t
any pos
All
thick
latter
concret
The
wooden
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The
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thick.
The
concret
paved
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The
piers
arch,
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forces
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from
crown
cal ba
edges
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not n

arches. As these forms were required to be carried high in the air, the support afforded by the beams, and consequently the lesser cost of scaffolding, doubtless quite offset the higher cost of the material.

The 15-in. beams and the 20-in. beams in line with them were firmly connected by heavy angle irons with four $\frac{15}{16}$-in. turned bolts, in order to constitute a tie between the two ribs of the arch, and prevent any possibility of a movement apart and split of the floor.

All the beams are completely encased in concrete, which is 3 in. thick over the top surface and around the bottom flanges. At the latter point, $\frac{3}{16}$-in. wire loops, 3 in. apart, are used to prevent the concrete from loosening.

The floor over the middle aisle is depressed to permit the use of wooden ties and cinder ballast for two street-railway tracks, and to afford also, if it should ever be required, depth for an underground trolley conduit.

The whole of the driveway floor surface below the paving is waterproofed with a coating of coal-tar roofing pitch, from $\frac{1}{8}$ to $\frac{1}{4}$ in. thick.

The depresssion is filled with cinders, over which is laid the 6-in. concrete paving base. The driveway, 40 ft. wide between curbs, is paved with asphalt, and the two sidewalks, 8 ft. 3 in. wide, are granolithic with steel-edged curbs.

The 2½-ft. longitudinal walls are connected at intervals over the piers and ribs of the approach arches, and over the ribs of the main arch, by 24-in. cross-walls, which serve as braces for the walls and as diaphragms or transverse bracing to transmit to the arch ribs the wind forces on the floor. The floor is separated by joints into sections to localize the contraction, and the interruption of its continuity prevents its action as a horizontal beam to transmit the wind stress to the main piers. Each section, therefore, is required to transmit its horizontal forces to the arch ribs. To provide the requisite stability in the long slender ribs of the main arch, they are flared 1 ft. 9 in. on each face, from the crown to the spring, increasing the width from 18 ft. at the crown to 21 ft. 6 in. at the spring. This flaring corresponds to a vertical batter on each face of 1 in 40 in a projected cross-section, and the edges of the ribs, therefore, are in planes parallel to and almost coinciding with the battered faces of the piers. It also causes a slight but not noticeable warp in the face surface, the ribs being made rectangu-

lar in section at every point in a radial direction. The thickness of the ring is 9 ft. 6 in. at the spring and 5 ft. 6 in. at the crown. The ribs of the approach spans are not flared—the width being 18 ft. throughout—and their faces are plumb.

The transverse walls do not extend over the space between the ribs, the latter being independent of each other, but the shafts of the piers at the ends of the main arch are connected by a transverse arch of 12 ft. clear span and 10 ft. wide, having embedded in it twelve 1-in. square steel bars diagonally crossed to act as transverse bracing for the shafts. The transverse walls on the ribs are 2 ft. thick throughout the bridge, except those over the west half of the main arch where they were increased in thickness to 2 ft. 4½ in., in order to provide sufficient weight to make the dead load on the arch symmetrical by counterbalancing the heavier load on the east half caused by the greater height of the spandrel piers due to the ascending grade over the bridge.

The abutments were especially designed to economize material and at the same time avoid too utilitarian an appearance. The thrust of the end arch ribs is utilized to balance the earth pressure; and the curvature of the wings, besides embellishing the structure, has the effect of broadening the foundation and thus giving the equivalent of a much thicker wall to resist overturning. This effect is increased by the construction of a buttress, 4 ft. thick, at the extreme end of the wing in a direction radial to the curve, or at an angle of 45° with the axis of the bridge. At the junction of the wings and the front wall, steel rods are embedded for the double purpose of giving cantilever value to the projecting wings, and preventing vertical cracks in the sharp re-entrant angles of the corners. Some of these rods are continued in a straight line past the middle of the curve of the wing and within 6 in. of the surface, in order to give, if it should be required, beam value to the wing between the abutment wall and the buttress at the end of the wing.

The east abutment is 64 ft. high, from foundation to coping, about 38 ft. of this being above the natural surface of the ground in front of it. The south wing of this abutment is built around a 30-in. water main, a hole 6 ft. in diameter being made in the wing to give free access to all sides of the pipe for repairs. The foundations of all the piers were required to be constructed over this pipe, and in each case an arch was formed over it to give 18 in. clear space around it, steel

FIG. 1.—FALSEWORK, WALNUT LANE BRIDGE.

FIG. 2.—MAIN ARCH CENTERING, WALNUT LANE BRIDGE.

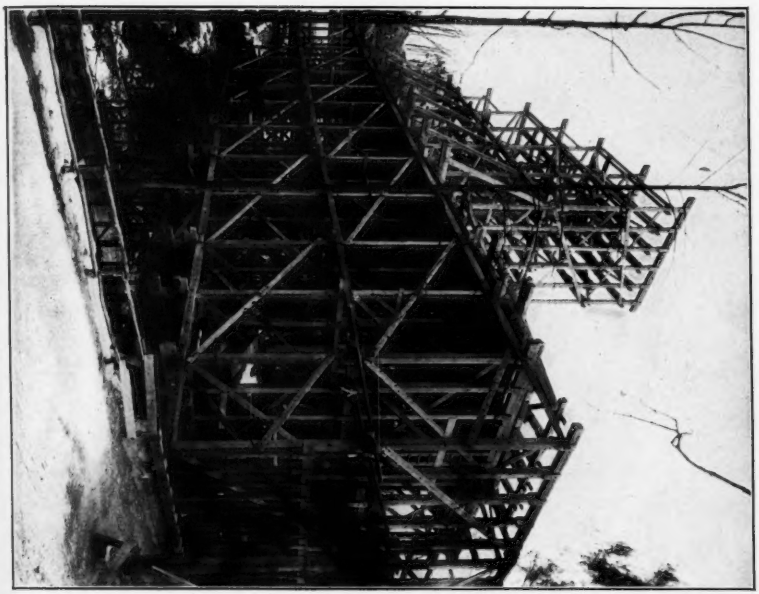


FIG. 1.—FALSEWORK, WALNUT LANE BRIDGE.

FIG. 2.—MAIN ARCH CENTERING, WALNUT LANE BRIDGE.

rods being embedded over the openings to resist any tendency to crack. The foundation of Pier 4 was extended because the pipe arch was near the end of the pier.

All piers, abutments, and wings, were founded on solid rock, the surface of which was found to be extremely rugged, requiring no stepping. In the foundations of each of the main arch piers two drill holes were sunk 8 ft. below the bottom to make sure of the solidity of the rock, and no crevices were found. The rock is a mica schist.

The open character of the spandrels on the main arch, with their numerous vertical joints, presented a condition quite different from that commonly found with solid spandrels of rigid walls and earth filling, where the load is distributed and the tendency to buck up is met with concentrated resistance. Here, the loads, except immediately over the crown, are all concentrated, and there is no stiffness in either walls or floor to concentrate resistance to buckling. Each panel delivers its own individual load, dead and live, at the same point always, and resists to only the same amount. The long ring rib, therefore, acts as a column, square-ended, throated in the middle, subject to varying eccentric loads, and made up of blocks accurately fitted together but not adhering, and, consequently, it is dependent for its stability entirely on its sufficiency of thickness to accommodate the varying center of pressure. Therefore, it became specially important to determine the extreme eccentricity of the pressure line by trying every possible condition of loading, both concentrated and distributed. The effects of six different conditions are shown by the stress diagram, Plate XLIV. The shape adopted for the arch is an intermediate between the extremes of eccentricity, coinciding very nearly with the line of dead or constant loading, with a slight arbitrary allowance at the spring for possible initial stress in the intrados from any shrinkage or settlement which might take place after keying, or from being keyed at a temperature above mean, as seemed likely to be the case. This shape was obtained by compounding segmental curves on three centers for each surface, as shown on Plates XXXIII and XLIV.

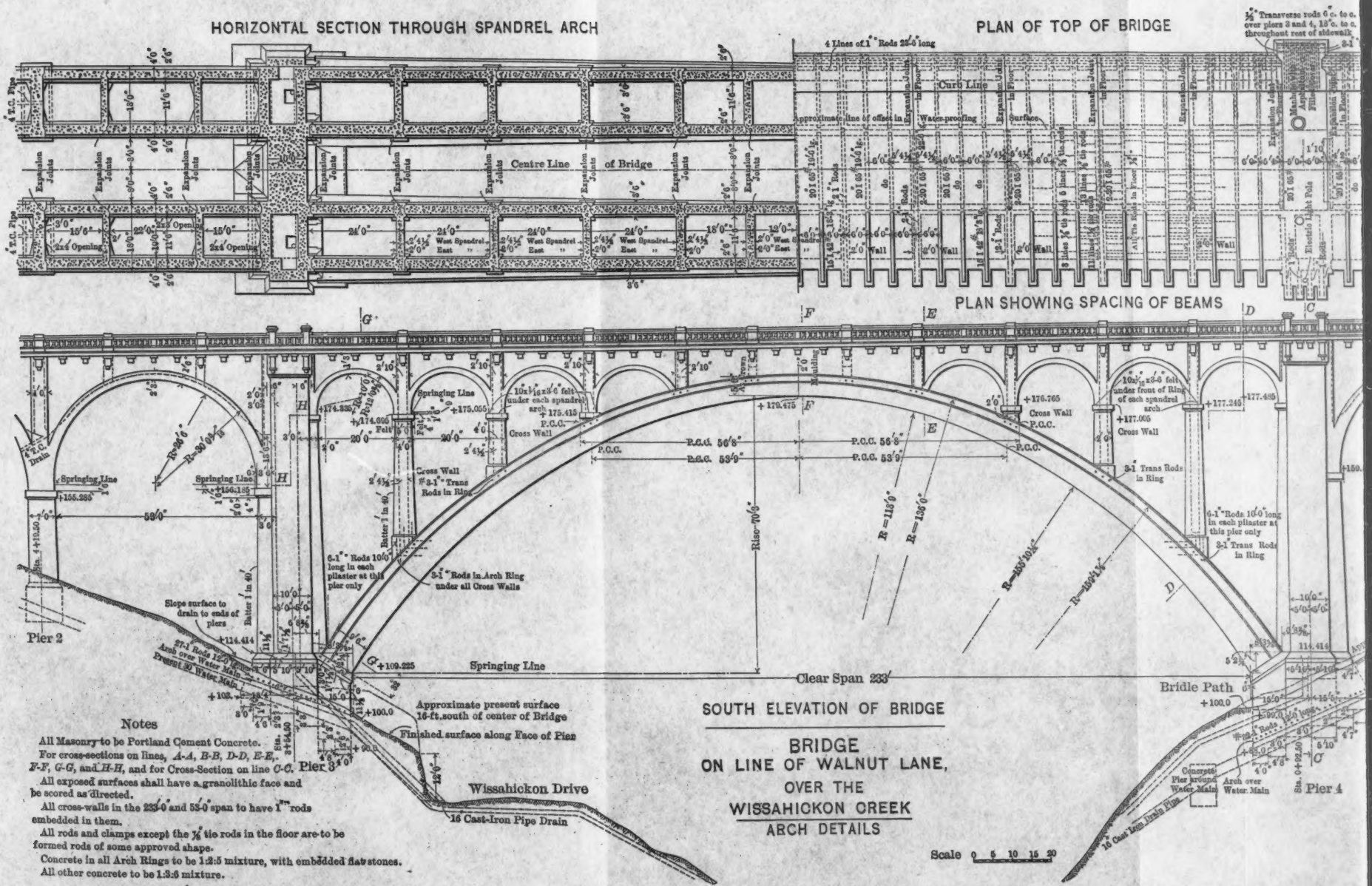
The arch was made square-ended, and continuous at the crown, in preference to hinging it. While pivotal bearings at the three points would have eliminated temperature stresses and reduced the likelihood of initial stresses from falsework settlement, the cost would have been

greater, and far less graceful lines would have resulted. The line of center pressure, if confined to pivots at springs and crown, would depart far from the mid-line of the ring at the quarters and require considerably greater thickness there than is needed with the flat-end bearing. The continuous taper in thickness as well as width of rib, from the abutments to the crown, is an important element of beauty in the design. In further pursuit of lightness in the appearance of the ring, a simple moulding was constructed on the outer face of each rib, tapering in width in proportion to the thickness of the ring.

The span of the arch, from center to center of bearings on the skewbacks, is 240 ft., and the rise of the mid-line is 70.5 ft.

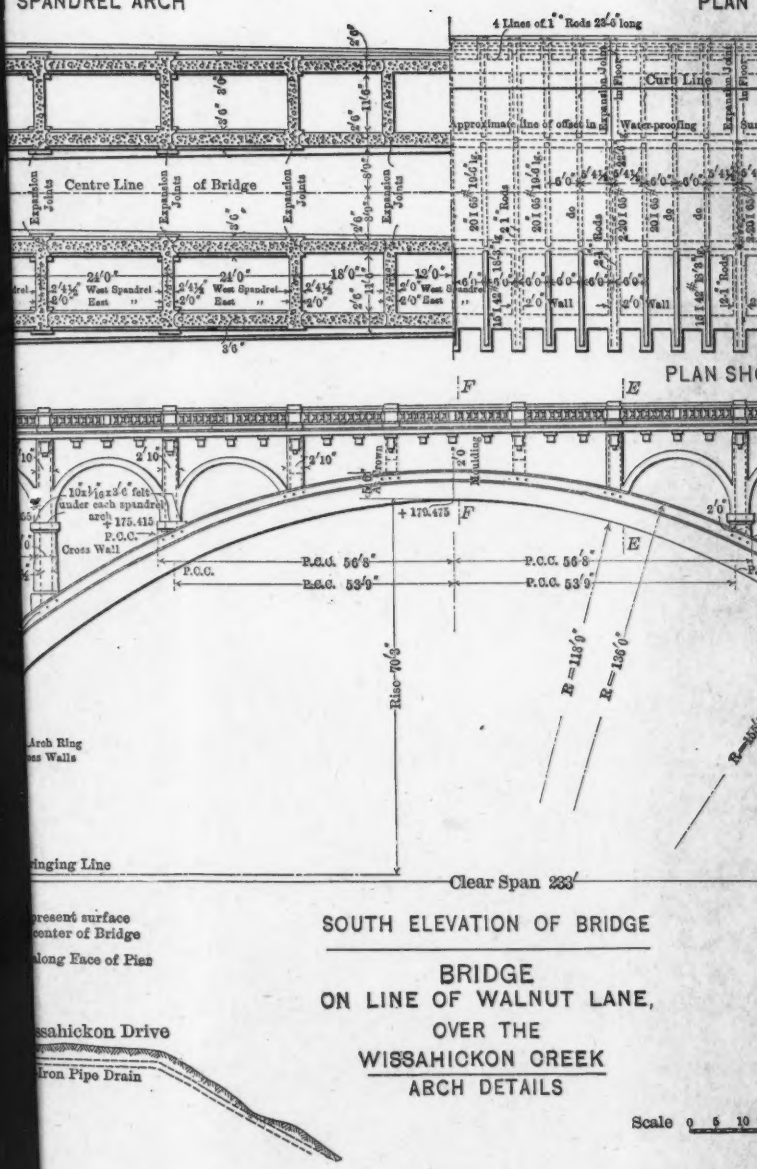
The maximum fiber stress appears to be at the spring, where there is the greatest theoretical eccentricity, because of the arbitrary allowance made for initial stress. This maximum is 356 lb. per sq. in., and the average at the same point is 167 lb.

The temperature stresses were computed in two ways, the results corresponding fairly. One method was by using the ordinary deflection formula for beams, and the other was by calculating the increase and decrease in length of extrados and intrados on the curvature resulting from the change in length of the ring and its confinement at the ends, the changes in length of fiber representing stress assumed to be in proportion to a modulus of elasticity of 2 000 000 in both compression and tension. Computations were based on a change of the general temperature of the arch of 40° Fahr., either way from the temperature at keying, which will change the length of the arch 1 in. and cause a change of 1 in. in the elevation of the crown. A rise increases the curvature at the crown, shortening its radius, and—because of the fixed ends—decreases the curvature of the haunches, lengthening the radius. This produces compression of the intrados at the crown—maximum at the middle—and of the extrados at the haunches—maximum at the spring—the effect being zero at the point of contra-flexure. This point was assumed to be at the *K* line of the stress diagram, Plate XLIV, which was computed by the Cain method, 82 ft. from the spring, and the substantial correctness of the locality was demonstrated by the close agreement of the bending stresses in the several divisions, the deformation of the arch being necessarily an automatic adjustment of the shape to produce uniformity of fiber stress in bending.



BRIDGE
ON LINE OF WALNUT LANE,
OVER THE
WISSAHICKON CREEK
ARCH DETAILS

Scale 0 5 10 15 20

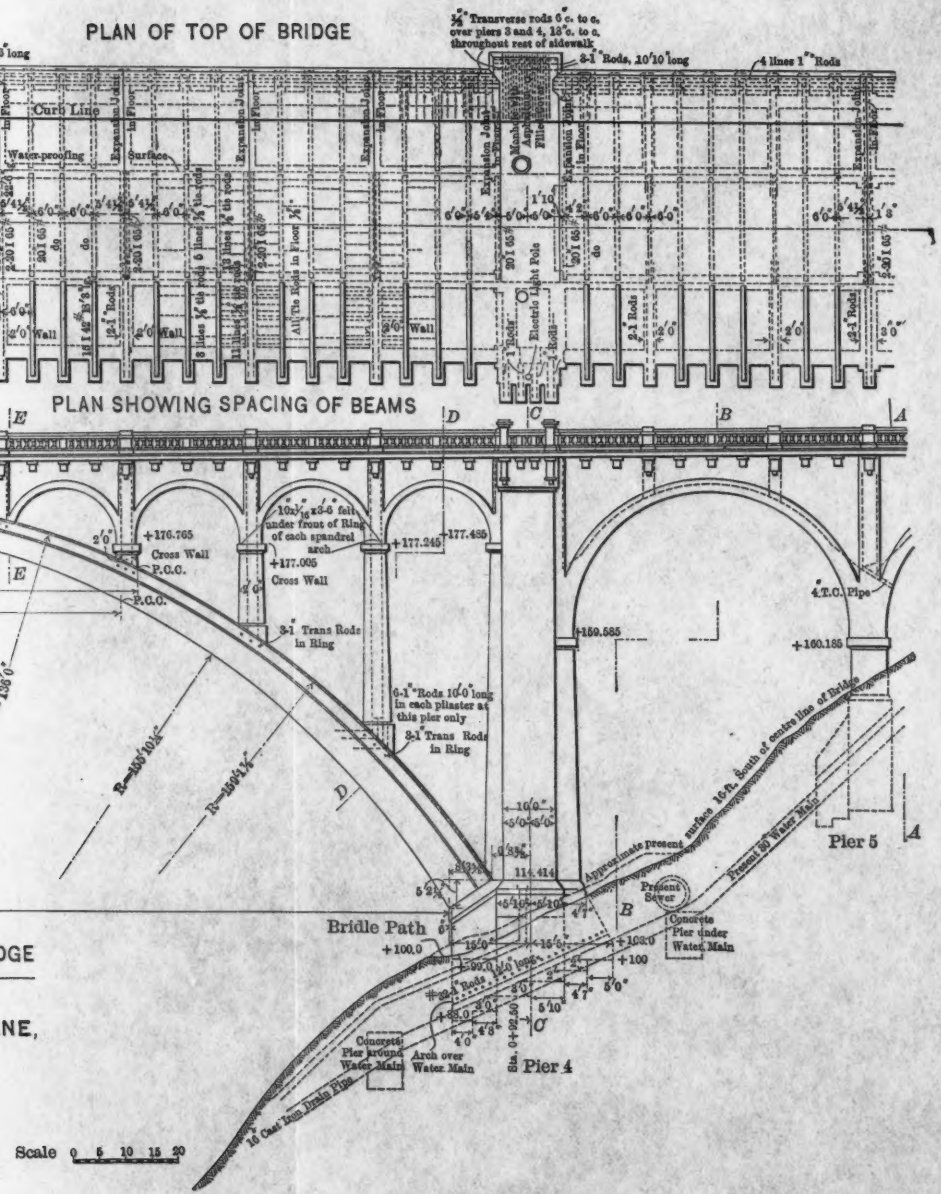


SOUTH ELEVATION OF BRIDGE

BRIDGE
ON LINE OF WALNUT LANE, OVER THE WISSAHICKON CREEK
ARCH DETAILS

Scale 0 5 10

PLATE XXXIII.
PAPERS, AM. SOC. C. E.
AUGUST, 1909.
WEBSTER AND QUIMBY ON
WALNUT LANE BRIDGE.



Measurements of the changes in elevation of the crown since the completion of the arch show a total difference between the extremes of winter and summer of 1⅛ in. which is equivalent to a change in length of arch ring of 1⅛ in. and a change in general temperature of the concrete of 45° Fahr. The actual temperature of the arch at the time of keying is not known for either rib. It is probable that the first one, because of some heat from crystallization and the atmospheric warmth of the mid-June season, was within 10 or 15° of the highest to be reached, and the second rib, keyed in November, was probably very near the mean temperature.

Some time after the completion of the arch rings, an electric resistance thermometer, of the standard type made by Leeds and Northrup, was embedded in one of the ribs on the mid-line near the spring, and weekly observations were made of the temperature of the heart of the concrete. A diagram showing the results of the changes in temperature and in the elevation of the crown of the arch is shown on Fig. 2. The heavy upper line gives the actual temperature in the arch; the dotted lines show the maximum, minimum, and mean weekly temperatures, as given by the United States Weather Bureau station in Philadelphia. The solid lower lines indicate the elevation of the two arch ribs. It will be observed that the total change of temperature in the center of the arch amounts to 42.8° Fahr., which gives a close check on both the assumptions used in the design, and on the computations of temperature changes made from the change in elevation. The close relation between the various curves in the diagram is also evident.

The concrete of the structure was of several classes, but in all classes—foundations, piers, abutments, walls, and balustrades—stones of as large size as conveniently workable were embedded. Some derrick stones were used in the foundations, but did not appear to be economical, because, under the specifications, a sufficient depth of concrete was required to be in place to half embed the stone, and it was found impracticable to get in as large a volume of embedded stone as by the use of one-man size. Besides, the monopoly of a derrick, during the time of placing a large stone, delayed the delivery of material and reduced the yardage of work accomplished. The quantity of stone thus embedded varied in different portions of the work, according to circumstances—the consistence of the concrete, the character of the sand

and coarse aggregate, the convenience of location, and the skill of the workmen. Observations of the relative quantities of stone and mixer concrete used have determined that in horizontal courses, such as are laid in piers and walls, the embedded stone constitutes from 30 to 40% of the total volume of the finished work, depending on the shape of the stone and the efforts of the laborers. The percentage secured in the arch, where all the stones were required to be placed radially, was from 25 to 30. The contractors were encouraged to use as large a percentage as could be embedded and fully flushed in the concrete, and they were of course desirous of using as much as possible, because its cost was less than that of the concrete displaced by it.

The use of this embedded stone is found to be economical in almost all classes of concrete work, even such as is reinforced with steel rods. Of course, care must be taken in reinforced work that stones do not ride upon rods in a manner to permit the formation of voids under the stones. The increase in compressive strength of all classes of concrete obtained by the embedment of large stones, as indicated by the tests of cubes, enhances the value of the material for almost if not quite all classes of work. The stones also very materially increase the shearing strength of the concrete, and generally reduce its cost.

The concrete of the ring of each 53-ft. arch was deposited continuously, was accomplished always in one day or less, and is of the same character and quality as that in the main arch—1:2:5 mixer concrete with flat stone embedded radially. The skewback was formed at the point of meeting of the arch ring with the transverse walls on the piers, the portion of the arch below this—called the umbrella—being constructed as a part of the pier.

The arches of this bridge, as constructed of this composite material, may be regarded as stone arches of a grade of workmanship superior to the ordinary rubble arch, because of the complete bedding of the individual stones. The increased compressive strength obtained by the embedment of the stone is attended, of course, with a considerable increase in the shearing resistance, and this quality is valued in consideration of the forces operating in the arch at the skewback, where eccentricity of direct stress produces actual shearing stress. The main arch ribs are practically columns, with more or less eccentric loading, and with varying and somewhat indefinite intensity of bearing stress at the ends. Concrete prisms or columns, when compressed to destruc-

Fig. 2.

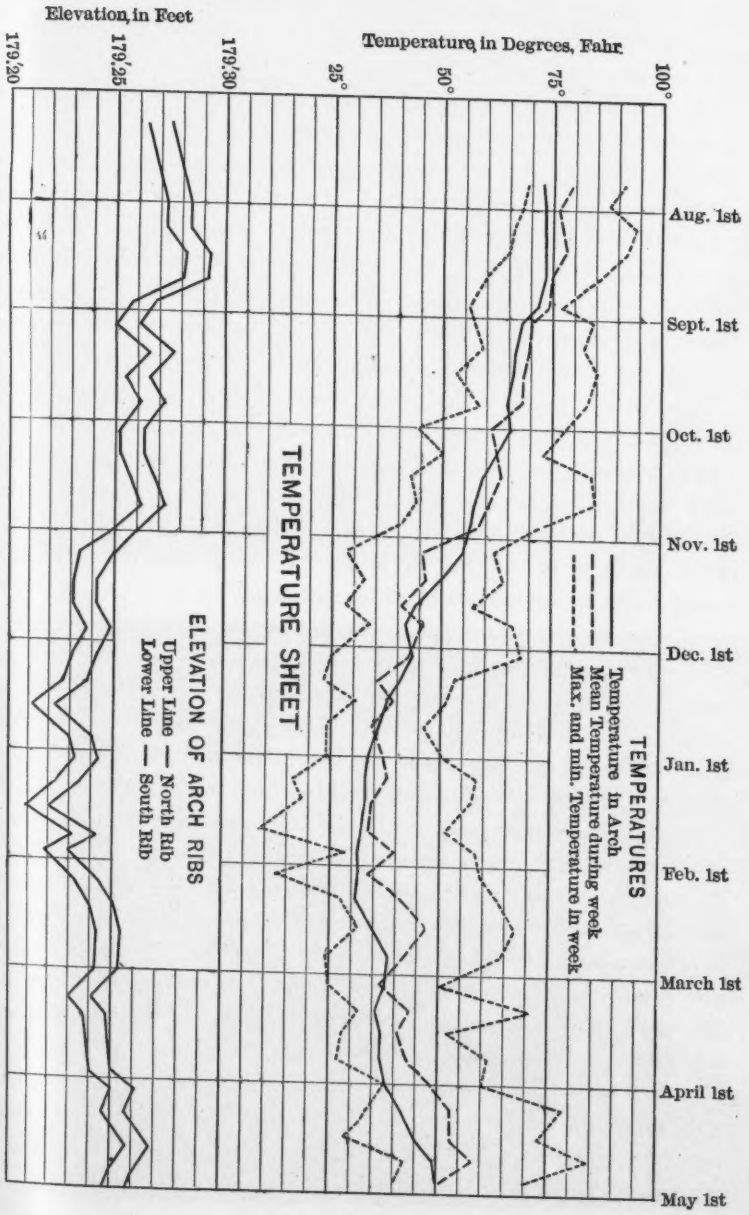

Elevation, in Feet
179.20
179.25
179.30
Temperature, in Degrees, Fahr.
25°
50°
75°
100°
Aug. 1st
Sept. 1st
Oct. 1st
Nov. 1st
Dec. 1st
Jan. 1st
Feb. 1st
March 1st
April 1st
May 1st
TEMPERATURE SHEET
ELEVATION OF ARCH RIBS
Upper Line — North Rib
Lower Line — South Rib
TEMPERATURES
Temperature in Arch
Mean Temperature during week
Max. and min. Temperature in week

FIG. 2.

tion, usually fail by shear, the angle approximating 39° with the axis of compression, and a force acting at any angle to the direction of the compression that causes the shearing will constitute a shearing force and hasten the failure. Such an angular force is produced by eccentricity of direct stress or by increments of loading and own weight. Tests of concrete model arches, made in order to determine questions of design, indicated that it is wise to treat the components of the direct arch stress at the abutment as so much shear acting in a beam. Regarded as such, the vertical component or reaction at the spring of the main arch ribs of this bridge amounts to 108 lb. per sq. in. of arch section in shear at the face of the skewback, which would be high for simple concrete, but is allowable for the hybrid material.

The mix of concrete used in the foundations and in the body of the piers and walls was 1 part Portland cement, 3 parts sand and 6 parts crushed stone. In the arches—the main and approach ribs and floor jack-arches—the mix was 1:2:5. The facing concrete or granolithic mixture of all showing surfaces, and also the body of the balustrade work was 1:2:3. It was all mixed so mushy that the large stone could easily be submerged in it by joggling.

The sand was required to be graded—grains varying from fine to coarse—and a yellow bank sand from New Jersey was accepted and used. Crushed stone was required to be graded in size from ¼ in. up, limited to 1½ in. generally, and to ¾ in. for arch and floor work. After using the 1½-in. size for a time the contractor preferred to use a 1-in. size limit because of the greater facility which it seemed to provide for embedding the large stone. The difficulty, however, with the 1½-in. stone was in obtaining a satisfactory grading—a proper admixture of all sizes from ¼ in. up—the practice of the quarries generally being to separate the 1½-in. stone from the smaller size. If, however, the run of the crusher up to the 1½-in. size can be obtained, the larger size will give equally satisfactory results and perhaps somewhat better concrete. The most thoroughly graded aggregate is found to produce, not only the densest and strongest concrete, but the smoothest working.

Every car lot of cement received was sampled after delivery at the work, and was tested in the municipal laboratory before being used. It was required to meet the standard specifications of the Bureau of Surveys of the City of Philadelphia, which are as follows:

FIG. 1.—WASHING THE SURFACE OF THE ARCH.

FIG. 2.—SOUTH RIB, WALNUT LANE BRIDGE, AFTER REMOVAL OF CENTERING.

FIG. 1.—WASHING THE SURFACE OF THE ARCH.

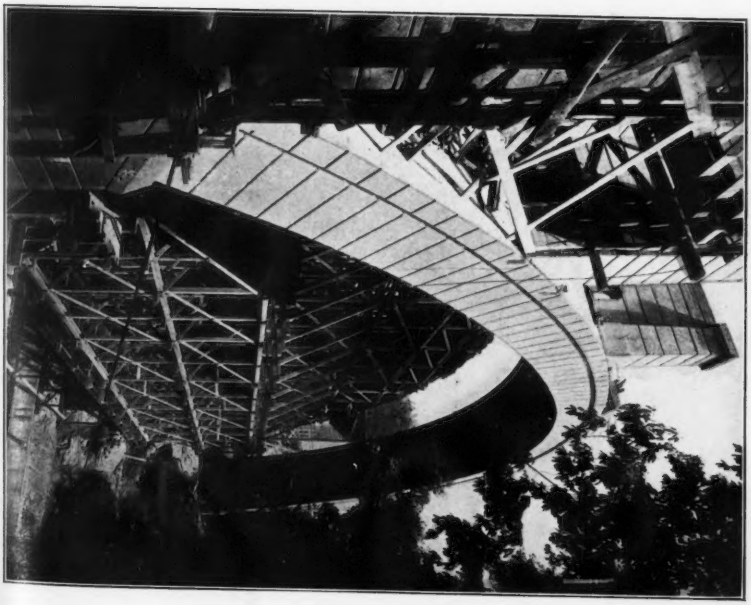


FIG. 2.—SOUTH RIB, WALNUT LANE BRIDGE, AFTER REMOVAL OF CENTERING.

"The acceptance or rejection of a cement to be used shall rest with the Chief Engineer, and will be based on the following requirements:

"Specific gravity not less than 3.1.

"Ultimate tensile strength:

7 days (1 day in air, 6 days in water)	500 lb.
28 days (1 day in air, 27 days in water)	600 lb.
7 days (1 day in air, 6 days in water) (1) one part cement to (3) three parts of standard quartz sand	170 lb.
28 days (1 day in air, 27 days in water) (1) one part cement to (3) three parts of standard quartz sand	240 lb.

"Fineness:

Residue on No. 100 sieve not over 8 per cent. by weight.

Residue on No. 200 sieve not over 25 per cent. by weight.

"Set: It shall require at least 20 minutes to develop initial set, and shall develop hard set in not less than 1 hour nor more than 10 hours. These requirements may be modified where the conditions of use make it desirable.

"Constancy of Volume: Pats of cement 3 inches in diameter, one-half inch thick at center tapering to thin edge, immersed in water after 24 hours in moist air, shall show no signs of cracking, distortion or disintegration. Similar pats in air shall also remain sound and hard. The cement shall pass such accelerated tests as the Chief Engineer may determine.

"Sulphuric anhydride (SO_3); not more than 1.75 per cent.

"Magnesia (MgO); not more than 4 per cent.

"Briquettes for testing shall be 1 sq. in. area of cross-section; sieves shall be of brass wire-cloth having approximately 9 800 and 37 500 meshes per square inch, respectively, the diameter of the wire being 0.0045 and 0.0023, respectively."

The accelerated test used consisted in making a small ball of neat cement paste, allowing it to harden 24 hours in moist air, and then boiling it for 3 hours.

For purposes of record and comparison, specimen 6-in. and 12-in. cubes of concrete, as taken from the mixer, were made by the inspectors at frequent intervals during the progress of the work, and tested in the city's laboratory. Standard briquettes were also made by the inspectors from the mortar extracted from the concrete. Table 1 shows the results of tests of both cubes and briquettes. A number of the cubes had embedded flat stones representing as nearly as practicable the character of the concrete made in the work.

| | | |

TABLE 1.—TESTS OF CONCRETE CUBES FROM WALNUT LANE BRIDGE.

Composition.	Age, 30 days.			Age, 60 days.			Age, 90 days.			Age, 120 days.			Age, 180 days.			Age, 1 year.		
	Compressive strength, lb. per sq. in.		Tensile briquettes.(1)	Compressive strength, lb. per sq. in.		Tensile briquettes.(1)	Compressive strength, lb. per sq. in.		Tensile briquettes.(1)	Compressive strength, lb. per sq. in.		Tensile briquettes.(1)	Compressive strength, lb. per sq. in.		Tensile briquettes.(1)	Compressive strength, lb. per sq. in.		Tensile briquettes.(1)
	6-in. cubes.	12-in. cubes.		6-in. cubes.	12-in. cubes.		6-in. cubes.	12-in. cubes.		6-in. cubes.	12-in. cubes.		6-in. cubes.	12-in. cubes.		6-in. cubes.	12-in. cubes.	
1:2:2 (2).....	2 462	2 111	341	.....	.....	....	.....	.....	....	.....	.....	....	.....	.....	....	.....	.....	....
1:2:5.....	1 841	2 055	197	4 236	.....	509	3 426	2 937	377	3 833	2 736	412	5 032	2 625	670	3 890	3 208	432
	2 089	.....	234	3 482	.....	442	4 684	2 875	....	3 702	.....	393	3 916	.....	438	3 664	2 694	435
	3 258	.....	349	4 009	.....	514	4 116	2 181	573	.....	.....	....	.....	.....	....	3 650	.....	299
	3 851	.....	597	4 452	.....	384	4 391	.....	504	.....	.....	....	.....	.....	....	3 001	....	333
	.....	.....	....	3 150	.....	434	3 209	.....	478	.....	.....	....	.....	.....	....	.....	.....	....
	.....	.....	....	3 164	.....	515	3 220	.....	489	.....	.....	....	.....	.....	....	.....	.....	....
	.....	.....	....	2 993	.....	454	3 959	.....	515	.....	.....	....	.....	....	....	.....	.....	....
Average.....	2 760	2 055	344	3 641	.....	465	3 858	2 664	489	3 768	2 736	403	4 474	2 625	554	3 526	2 951	375
1:2:5 (2).....	2 372	2 111	406	(3)4 111	3 500	843	(3)4 655	2 694	681	.....	3 583	....	4 671	3 805	689	5 072	4 111	455
	2 010	2 861	496	3 512	.....	664	3 918	3 986	690	.....	.....	....	3 760	.....	659	4 375	4 264	477
	1 634	3 174	535	.....	.....	....	.....	4 320	....	.....	.....	....	4 201	.....	611	.....	.....	....
	1 948	.....	646	.....	.....	....	.....	2 806	....	.....	.....	....	.....	.....	...	.....	.....	....
	1 870	.....	369	.....	.....	....	.....	2 764	....	.....	.....	....	.....	.....	....	.....	.....	....
	2 406	.....	375	.....	.....	....	.....	.....	....	.....	.....	....	.....	.....	...	.....	.....	....
	2 395	.....	411	.....	.....	....	.....	.....	....	.....	.....	....	.....	.....	....	.....	.....	....
	2 712	.....	451	.....	.....	....	.....	.....	....	.....	.....	....	.....	.....	....	.....	.....	....
	2 689	.....	444	.....	.....	....	.....	.....	....	....	.....	....	.....	.....	....	.....	.....	....
Average (2)..	2 226	2 715	459	3 812	3 500	754	4 287	3 314	686	.....	3 583	....	4 211	3 805	653	4 724	4 188	466
1:3:6	1 091	1 440	385	2 696	2 033	291	3 568	2 097	676	3 613	1 833	238	2 683	3 479	333	3 676	.....	375
	1 993	(5)1 354	394	3 119	2 153	398	3 507	2 472	637	2 157	2 403	362	1 817	2 292	245	3 023	.....	444
	1 754	1 778	392	3 223	2 097	448	3 301	1 777	531	2 414	2 056	443	2 447	2 208	367	.....	.....	....
	2 018	1 916	218	2 704	1 896	387	2 888	1 743	526	2 118	2 861	225	2 453	2 833	273	.....	.....	....
	2 016	1 986	436	3 382	1 799	407	3 679	1 833	670	3 111	.....	339	2 168	2 708	421	.....	.....	....
	2 456	(9)1 694	539	4 017	1 903	481	2 536	2 861	469	2 962	.....	294	3 176	.....	393	.....	.....	....
	(4) 850	1 924	230	2 249	1 687	448	3 097	2 125	491	3 465	.....	457	2 908	.....	290	.....	.....	....
	1 678	1 888	363	3 151	2 083	490	2 829	1 666	554	4 326	.....	576	2 586	.....	236	.....	.....	....
	1 669	.....	253	3 215	1 861	603	2 234	1 916	358	4 256	.....	458	5 074	.....	498	.....	.....	....
	1 544	.....	278	2 944	1 903	505	2 317	2 000	485	.....	.....	....	4 928	.....	304	.....	.....	....
	(6)1 325	.....	176	2 029	.....	306	2 391	1 757	379	.....	.....	....	2 917	.....	448	.....	.....	....
						336	2 436	2 486	466	.....	.....	....	5 164	.....	423	.....	.....	....
													4 143	.....	200	.....	.....	....

It will be observed that:

(1.) The 12-in. cubes have a strength of about 80% of the 6-in. cubes;

(2.) The 12-in. cubes containing large stones embedded in them average from 25 to 35% stronger than plain cubes;

(3.) The strength of the concrete shows a fairly regular increase with age;

(4.) The tensile strength of the mortar taken from the concrete is from one-fifth to one-eighth of the compressive strength of the concrete;

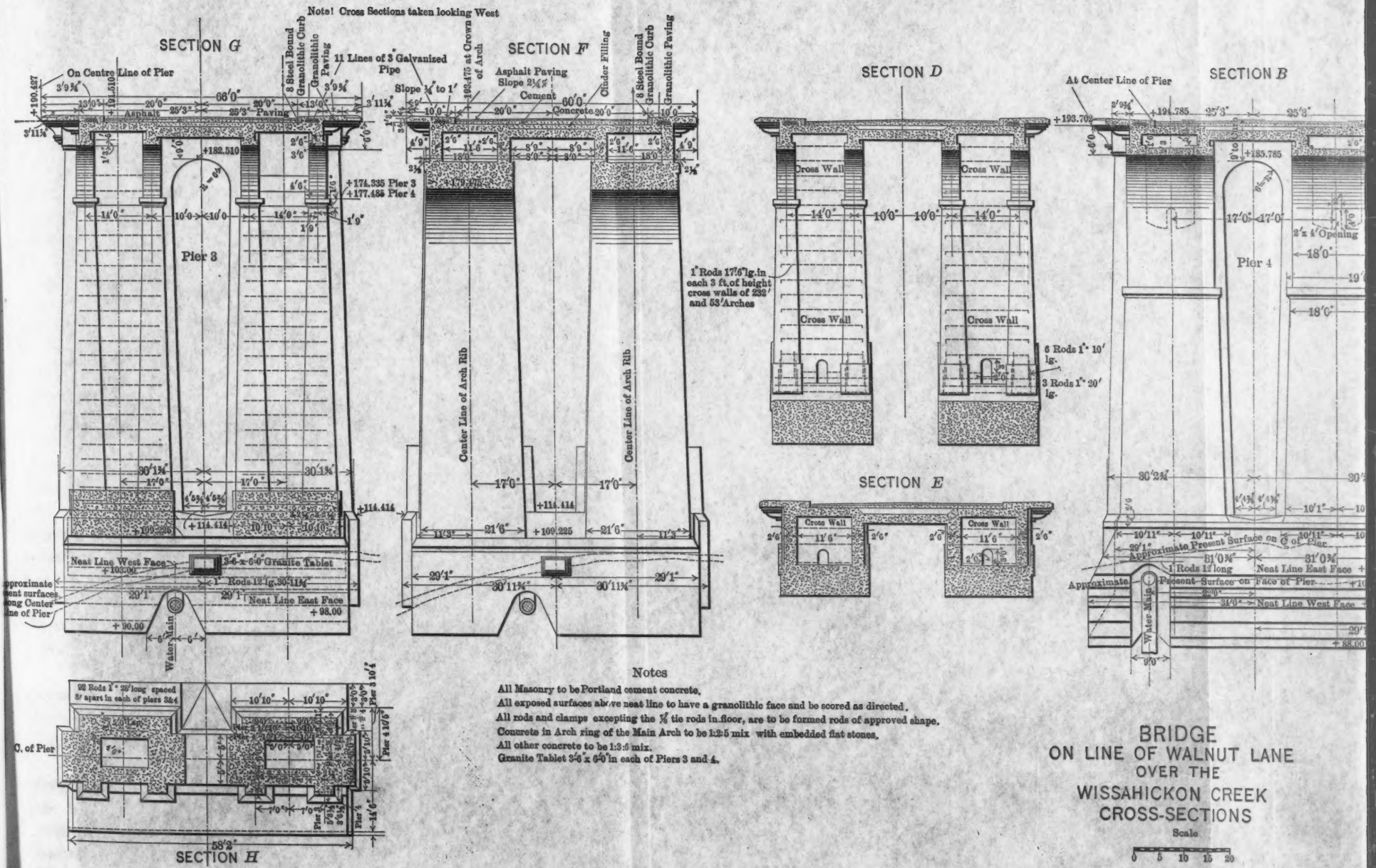
(5.) The strength of the 6-months old concrete may be assumed to be at least 2 000 lb. per sq. in.

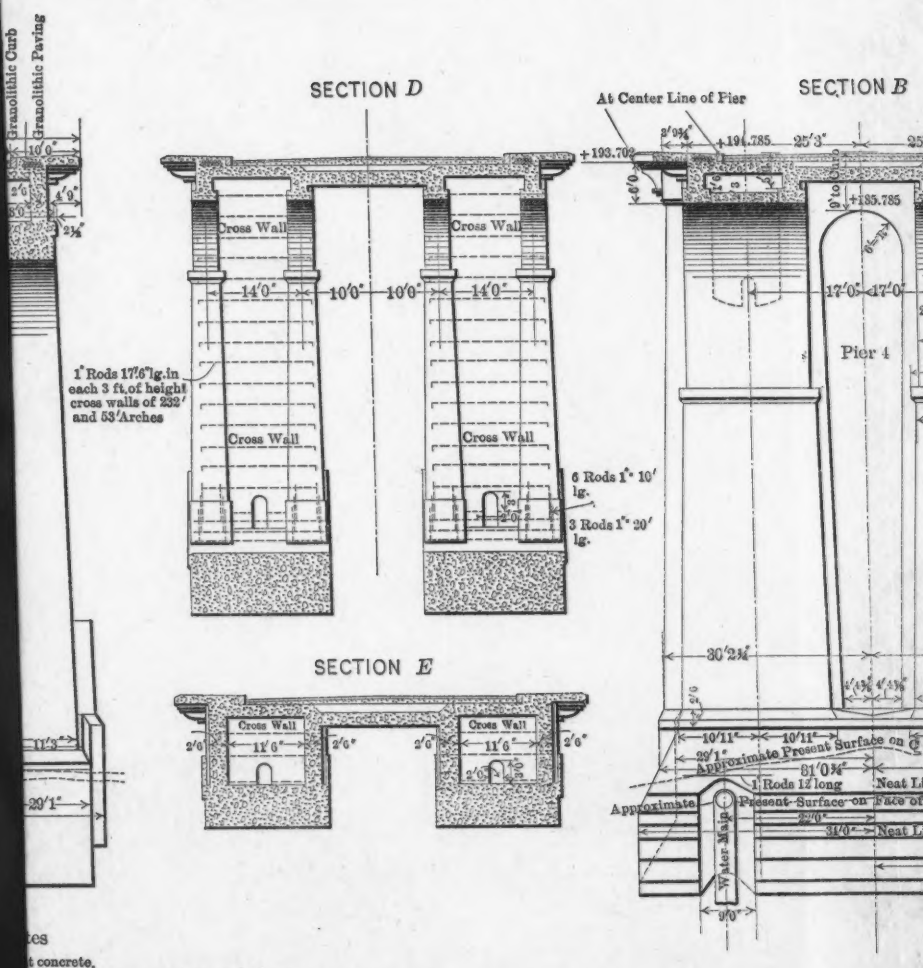
The crushed stone used in the early part of the work and throughout the construction of the first main arch rib was a very hard black shale of high shearing strength and well graded in size, but was so sharply angular that it was gritty in the concrete and was thought to be less smooth working than limestone. Accordingly, a change was made to crushed limestone, with very satisfactory results. The composition and properties of these stones were as follows:

		Limestone.	Black shale.
Composition:	Passing No. 10 sieve	1.9%	1.5%
	" ¼-in. sieve	15.5%	12.5%
	" ½-" "	44.6%	34.7%
	" ¾-" "	77.6%	70.4%
	" 1-" "	96.8%	94.8%
	Retained on 1-" "	3.2%	5.2%
	Weight per cubic foot	98.9 lb.	90.9 lb.
	Specific gravity	2.84	2.66
	Percentage of voids	44.1	45.1

The embedded stone used throughout the work is known as "Conshohocken" stone—a laminated silicious rock of very high compressive strength—which comes from the quarry in flat shape and with tapering edges which facilitated the embedment of the stones in a radial direction in the arch rings, and permitted close packing with good bonding in the horizontal courses of the piers, etc.

The showing faces of the concrete throughout the bridge were made with a fine concrete or granolithic mixture, using for all except the balustrades a crushed black shale which had passed a ⅜-in. screen and


Note! Cross Sections taken looking West
SECTION G
SECTION F
SECTION D
SECTION B
SECTION E
SECTION H
On Centre Line of Pier
Asphalt Paving
Pier 3
Pier 4
Cross Wall
Center Line of Arch Rib
Neat Line West Face
Neat Line East Face
Water Main
Approximate Present Surface on ℄ of Pier
1" Rods 17'6" lg. in each 3 ft. of height cross walls of 232' and 53' Arches
6 Rods 1" 10' lg.
3 Rods 1" 20' lg.
Notes
All Masonry to be Portland cement concrete.
All exposed surfaces above neat line to have a granolithic face and be scored as directed.
All rods and clamps excepting the ¾" tie rods in floor, are to be formed rods of approved shape.
Concrete in Arch ring of the Main Arch to be 1:2:5 mix with embedded flat stones.
All other concrete to be 1:3:6 mix.
Granite Tablet 3'6" x 6'0" in each of Piers 3 and 4.
BRIDGE
ON LINE OF WALNUT LANE
OVER THE
WISSAHICKON CREEK
CROSS-SECTIONS
Scale
0 5 10 15 20



tes

t concrete.

ne to have a granolithic face and be scored as directed.

¾" tie rods in floor, are to be formed rods of approved shape.

Arch to be 1:2:5 mix with embedded flat stones.

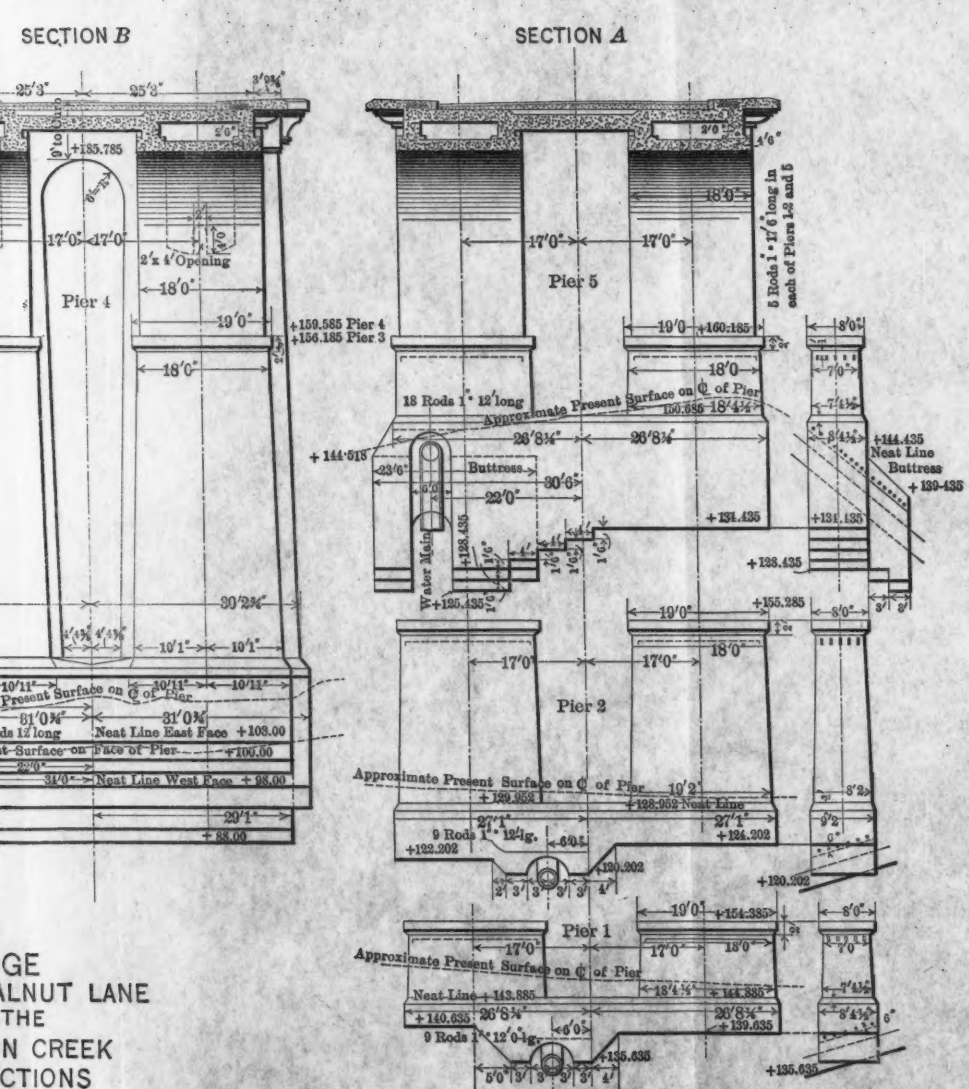
Piers 3 and 4.

BRIDGE
ON LINE OF WALNUT LANE
OVER THE
WISSAHICKON CREEK
CROSS-SECTIONS

Scale

0 5 10 15 20

PLATE XXXV.
PAPERS, AM. SOC. C. E.
AUGUST, 1909.
WEBSTER AND QUIMBY ON
WALNUT LANE BRIDGE.



GE
ALNUT LANE
THE
N CREEK
CTIONS
ale
0 15 20

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was cleaned of dust. For the balustrades the crushed stone was granite, ¼-in. size. The mixture used was quite wet, and was deposited against the face forms by pouring from the spout of a coal scuttle, and the body concrete was laid against it in such a manner as to maintain always a coating of the mixture against the forms. As the mixture and the concrete were both of wet consistency, they readily united to form a monolithic mass. No case has yet been observed of the separation of the facing mixture from the body concrete. The face of the work throughout was finished by removing the forms as soon as practicable after the concrete had set and while it was still friable, and then with brushes and water washing off the surface sufficiently to expose the grit or aggregate of the facing mixture. The forms were generally removed on the day following that on which the concrete was deposited. In cold weather the forms were required to remain from two to three days to permit the concrete to harden sufficiently to maintain itself safely after the removal of the forms.

This process of finishing produces a surface which very closely resembles stone in texture, and, as it removes the cement film which segregates against the forms, all likelihood of surface cracking is eliminated. The tone or color of the surface, of course, depends on the aggregate used in the facing mortar. The black shale grit and the yellow bank sand used for facing the body of this bridge produce the general color and tone of Eastern granite. The mixture used in the construction of the balustrades, which constituted both the body and the face, was made with crushed granite screenings in place of the black shale grit used in the remainder of the work, and a pleasing contrast in tone was thereby obtained.

The use of this fine concrete for surfacing was for the twofold purpose of obtaining a more durable material, it being richer in cement, and a more uniform texture than would result from washing a concrete of coarser aggregate.

To facilitate the removal of the forms for finishing the surface, they were constructed with the uprights or studs set about 8 in. from the face of the work, and the planks of the forms, which were in width equal to the depth of the courses, were secured in place with easily removable cleats tacked to the uprights. On the top and bottom edges of the planks were fastened bead strips, 2½ in. wide, projecting

beyond the planks into the concrete to cause imprints. Courses were finished at the apex of the top bead, and, when the forms and planks were removed on the following day to permit the washing of the surface, they were raised one course, so that the bottom bead fitted into the half imprint left by the top bead. The work was thus built up course by course, in the smaller portions several courses being laid at one operation. No vertical beads were used, and no effort was made to imitate ashlar stone construction.

As the soffits of the arches could not be exposed for washing until the centers were removed, and the surface was then too hard to be removed by a brush, the impression of the lagging was removed by tool dressing. Pneumatic bush-hammers were used from a swinging scaffold, and the work was done before the floor was laid, so that the scaffold could be supported from out-looker timbers on top of the spandrel walls.

The construction of the main arch ribs was by blocks or voussoirs of a size limited to 75 cu. yd. each, always in pairs simultaneously—one on each side—so that the horizontal thrusting effect on the centering would be kept balanced.

The specifications prescribed that each section or block was to be constructed continuously from start to finish without cessation, in order to avoid joints, but in several cases it was found to be impossible to complete the section before dark, and forms were then erected to give to the day's work a radial face. Generally, it was found possible to complete a pair of blocks within daylight of one day—an output of about 150 cu. yd.

The number of large blocks was twenty-four—in twelve pairs—placed in the order of their numbers as shown by Fig. 3, with seventeen small blocks or keys. The first two cast were adjacent to the crown key, thus reversing the usual order of procedure in the construction of masonry arches, although it has been customary to weight down arch centers at the middle with a temporary load to prevent the rising at the crown caused by the thrust of the arch at the haunches. The arrangement was such as to leave seventeen small spaces for keying, the size of these spaces varying from 2 ft. 8 in. for the lower key to 2 ft. 0 in. for the crown key.

The method of construction used—building the voussoirs independently, scattered over the centering, to be joined later by the keys

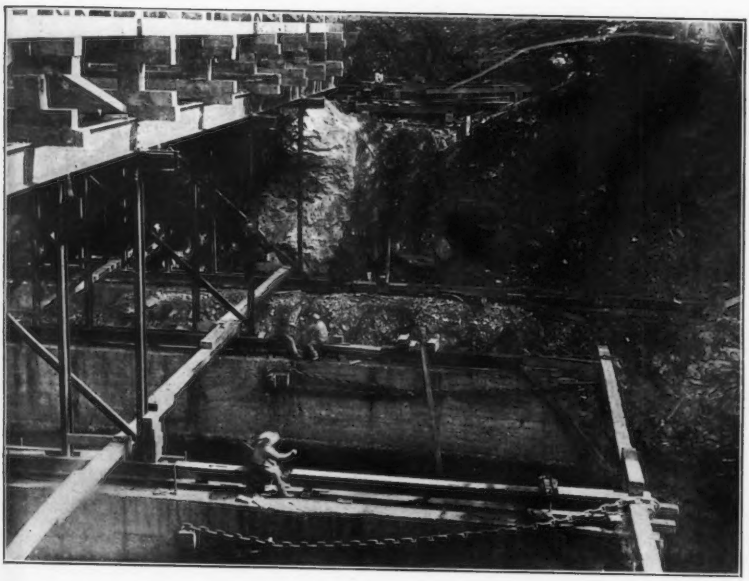


FIG. 1.—MOVING THE CENTERING, WALNUT LANE BRIDGE.

FIG. 2.—VOUSSOIRS OF NORTH RIB, WALNUT LANE BRIDGE.

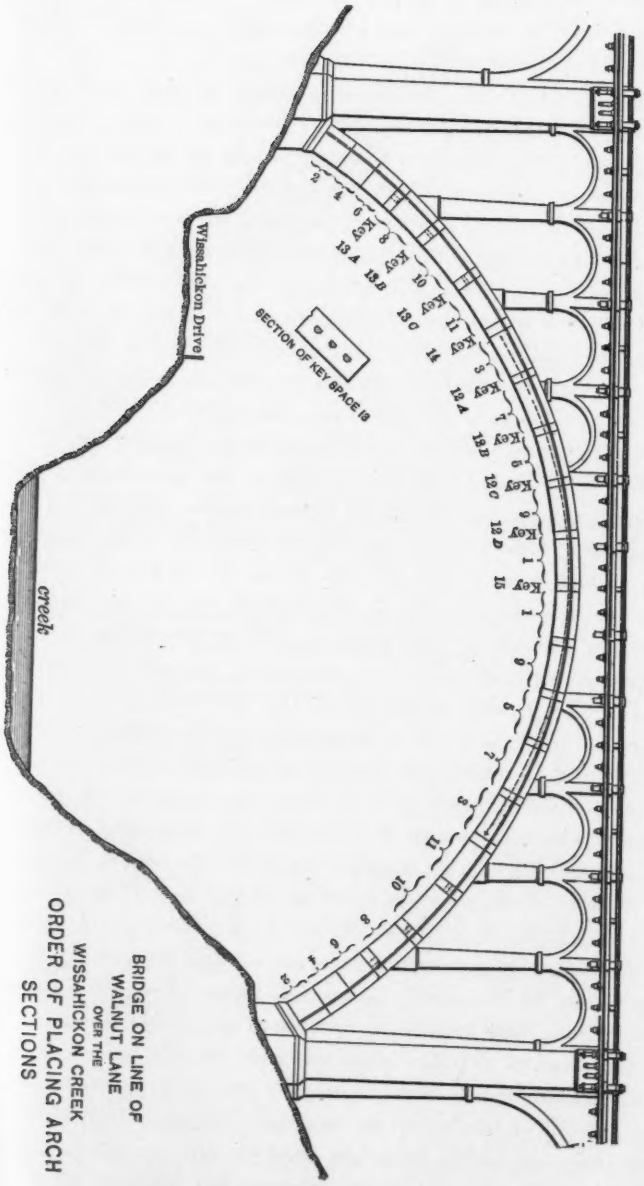

Wissahickon Drive
creek
SECTION OF KEY SPACE 13
BRIDGE ON LINE OF
WALNUT LANE
OVER THE
WISSAHICKON CREEK
ORDER OF PLACING ARCH
SECTIONS

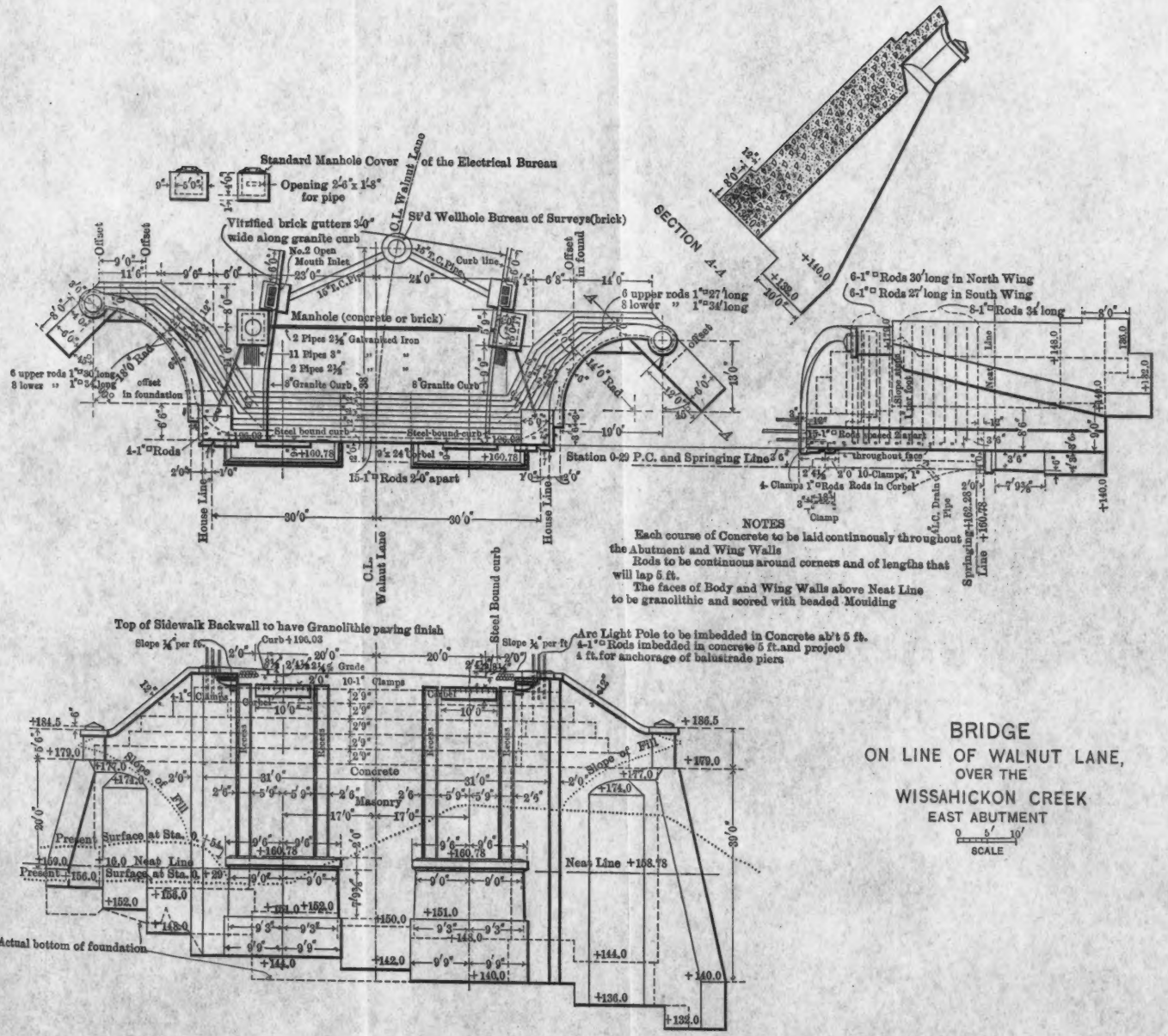
Fig. 3.

—required extreme care in the building of the forms, but the work was done very accurately, and all the edges of the ribs are in good line, and are entirely without kinks or waves.

The purpose of this method of construction was to distribute the load over the centering and permit the centering to attain its maximum deformation before laying, in order to prevent the occurrence of cracks in the arch, a condition which has generally been experienced in the construction of both stone and concrete arches under the usual method of beginning at the spring and proceeding continuously up to and keying at the crown. The utility of the method was demonstrated long before the completion of the first ring. It proved to be entirely successful, and although each main arch rib has in it forty joints, which are without sufficient adhesion of surfaces to give tensile value, there has not yet been discovered any disposition to open anywhere as the result either of loading or of temperature changes.

Some of the individual voussoir blocks on the steeper portions of the centering were required to be secured against sliding down hill on the lagging. This was accomplished in the case of the third pair of blocks—which were at the quarter points—by means of three 1¼-in., round, upset rods, made up to the required length with turnbuckles, and having 12 by 12 by ⅝-in. plate washers at the extreme ends. These rods pass over the crown at the mid-line of the arch, and Blocks 1, 3, 5, 7, and 9, were cast around them, the end washers being embedded in Block 3. No indication of movement of any of these blocks on the lagging could be found, and the one or two that became stressed must have been called into action by a slight movement of the joists. For the three lower key spaces on each side, the separation of the blocks was maintained by small concrete struts on the mid-line of the arch ring, three in each space—18 by 24 in. in the first, 15 by 20 in. in the second, and 12 by 18 in. in the third. As these struts must be built in with the key, they necessarily form an integral part of the arch, and as they must be subject to compressive stress in resisting the sliding of the sections, they must have, because of this initial stress, somewhat excessive pressure on them after the arch receives its full load. The load to be sustained by these struts was in a large measure dependent on the amount of friction on the lagging. Assuming that, if the blocks should move enough to break the bond of adhesion of the cement to the lagging, there would be no friction, the pressure on the struts

PLATE XXXVII.
PAPERS, AM. SOC. C. E.
AUGUST, 1909.
WEBSTER AND QUIMBY ON
WALNUT LANE BRIDGE.



BRIDGE
ON LINE OF WALNUT LANE,
OVER THE
WISSAHICKON CREEK
EAST ABUTMENT

0 5' 10'
SCALE

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would
concret
compre
be subj
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The m
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would be about 300 lb. per sq. in. of gross area. Any shrinkage of the concrete body of a key would entail the shortening of the struts by compression from arch stress before the main body of the key would be subjected to pressure. A shrinkage of as much as $\frac{1}{32}$ in. in the key would require for the corresponding compression of the struts a pressure of about 2 000 lb. per sq. in. if the modulus of elasticity of the concrete be 2 000 000 lb. per sq. in. This pressure on a small enclosed area would not cause any injurious effects on the material. The manner in which the keys were constructed, however, practically eliminated shrinkage, being in the nature of underpinning work. The mushy concrete was deposited in shallow layers, and more or less wedge-shaped stones were worked and rammed into it. The only visible effect of shrinkage in any of the keys was at the extreme top of the upper joint, where a very fine hair-line crack was distinguishable. In many not even this could be found because the concrete was banked and dammed up higher than the top edge, on the sink-head principle, this extra body being dressed off after hard set.

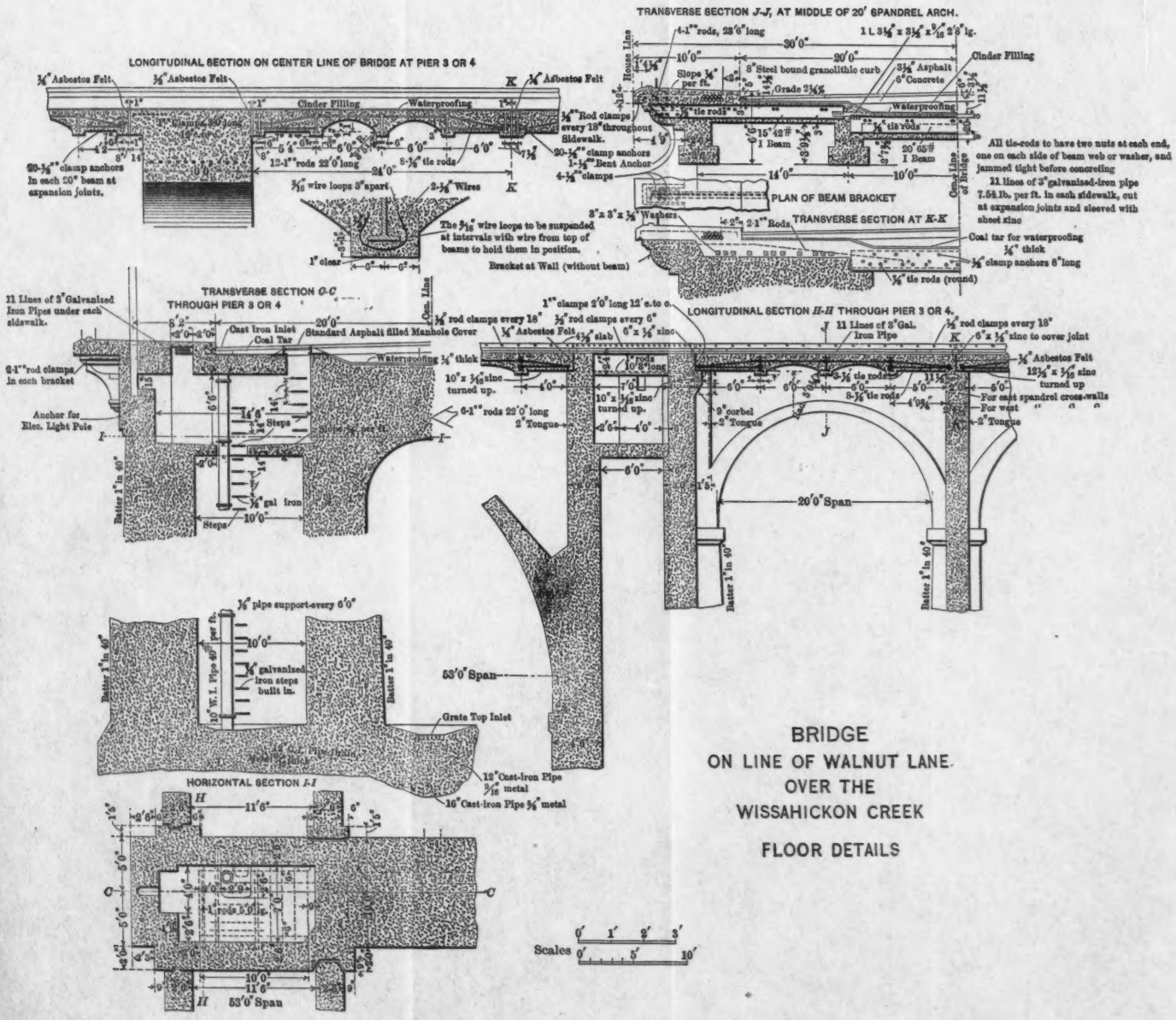
The anticipated hinge action of these struts developed as the load on the center increased, and it was demonstrated by their progressive cracking as the successive blocks were cast. The crack in each case opened on the upper or outer side of the struts, extending generally to within 6 in. of the under side, thus reducing the actual area for service to a fraction of what was provided, but it was evidently ample for the actual requirements. No special effort was made to grout up these openings, and any failure of the liquid of the concrete to penetrate and fill them would of course leave some small portion of the arch section unflushed and void, but the area thus lost was trifling, and, moreover, was at the least important part of the arch—the mid-line.

The theory of the construction of this arch, involving the principle of numerous keys and temporary hinges, logically called for a key space at the springing line, but it was omitted there because the satisfactory construction of a key would have been difficult, and the struts necessary for maintaining the space during the construction of the remainder of the arch would have been large enough to interfere somewhat with the proper deposit and compacting of the concrete of the body beneath the struts. The three lower blocks, therefore—Blocks 2, 4, and 6, Fig. 2—were cast in contact, and thus formed practically one section about 25 ft. long, directly on the skewback and

with its center of gravity about 4 ft. in front of the face of the abutment. The correctness of the theory referred to—that a key was necessary at the spring—was demonstrated when Block 8 was cast. The pressure on the falsework, at this stage of the construction, caused sufficient yielding of the centering to permit the heavy 25-ft. section to tilt forward and separate from the skewback at the outer side while bearing on the inner or lower edge. This action occurred at all four of the skewbacks, the cracks thus formed varying in size of opening from $\frac{1}{8}$ in. on one skewback to $\frac{3}{8}$ in. on another at the extrados of the arch, and extending to within about 12 in. of the intrados.

As the construction of the arch on the sections as tilted would have concentrated all the arch pressure on a fraction of the area of the ring at the skewback, it was necessary to push these sections back until their bearing was uniform over the whole width of the skewback. This was accomplished with screw-jacks and wedges acting under the joists and against the horizontal timbers of the falsework, and it is believed that, because of this adjustment, practically all initial stress in the arch ring was avoided. The distribution of the voussoir blocks over the centering, before constructing any of the keys, gave to the falsework almost all its load, and produced the maximum deformation in it before the keys closed the ring. The entire absence of openings in any of the joints between keys and sections, and of any cracks in the sections at any subsequent stage of the erection, is proof of the fact that the line of pressure was always within the middle third of the depth of the ring, for none of the joints have any tensile resistance to opening.

One of the ribs was subjected to an unintentional but interesting test of its stability by inadvertent unsymmetrical loading during the construction of the spandrel walls, which work was done after the centering had been removed. The work of construction over the arch was required to be carried on simultaneously on opposite sides, in order to avoid unsymmetrical loading, but, through a combination of circumstances, the rule was neglected for a part of a day, and a weight of about 100 tons of concrete was deposited in the spandrel walls of one section immediately over one of the quarters without any balancing load on the opposite quarter. As soon as the condition was discovered a careful examination was made of all the joints of the arch, but not the slightest evidence of distortion could be perceived. A pressure



BRIDGE
ON LINE OF WALNUT LANE OVER THE WISSAHICKON CREEK
FLOOR DETAILS

diagram, covering this condition, was made subsequently, and the center of pressure was found to be within the middle third of the arch ring. This is accounted for by the great weight of the arch ring itself.

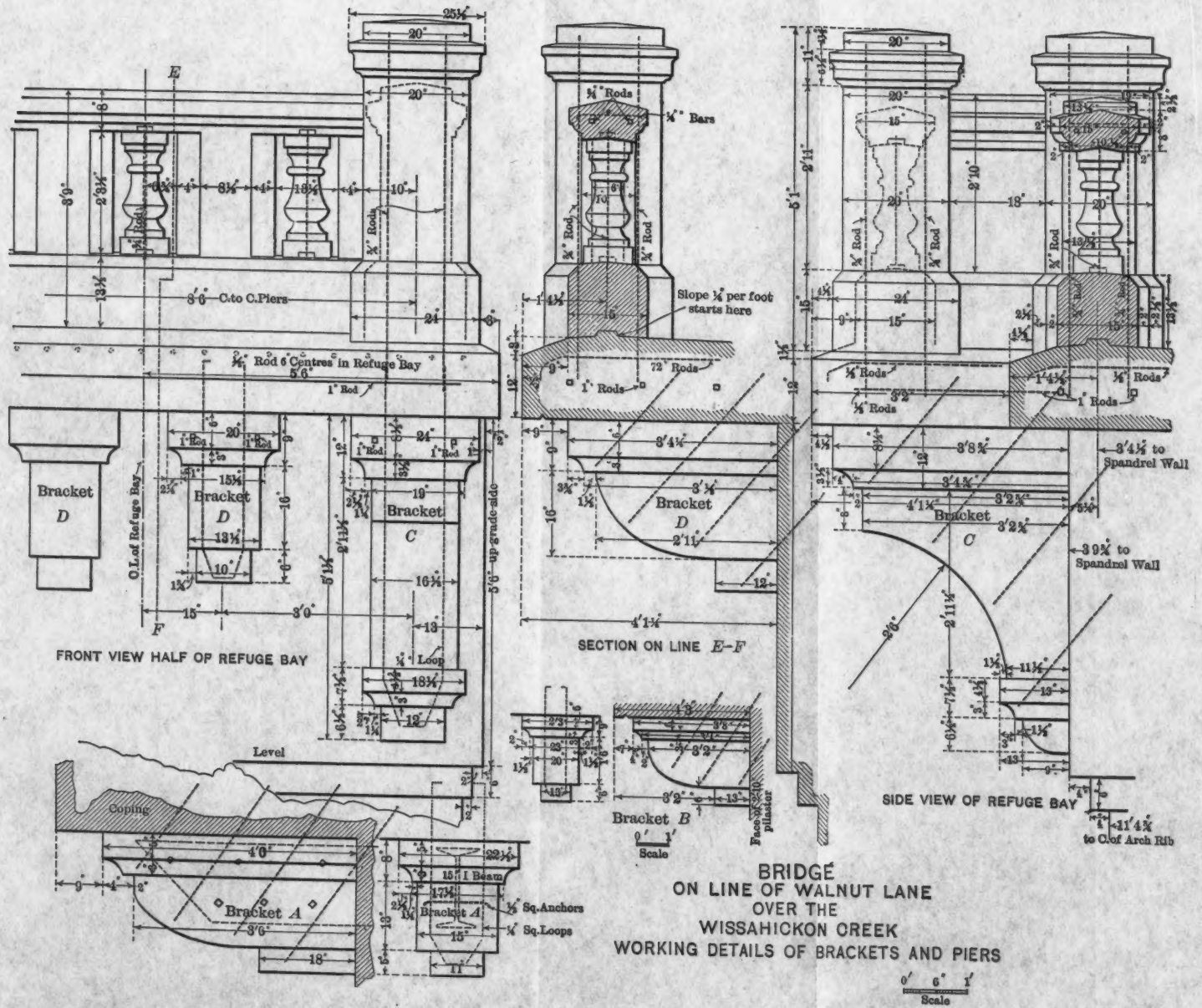
The centering was required to be maintained under the completed arch ring for at least one month. The first ring was keyed in a season of rising temperature (June 7th), and the wedges of the centering were struck one month later. The second arch was keyed in cool weather (October 17th), and the support was continued for 6 weeks. The stresses in the woodwork were high enough, apparently, to produce progressive deflection, so that, during the time of loading and subsequent seasoning, the falsework compressed sufficiently to lower the crown $3\frac{3}{4}$ in., this amount being the same for both ribs. In anticipation of this compression, the centering was originally set higher than the plan elevation. During the construction of the keys the woodwork was saturated with water as far as practicable by flowing water upon the bents through holes in the lagging, and the water was shut off as soon as the keys were completed, the purpose being to obtain the advantage of the shrinkage of the wood in drying out to withdraw gradually from the arch some of the support of the centering, and thereby facilitate the striking of the wedges. The utility of this process is doubtful when as high stresses are put in the wood as was done in this case, for it is probable that the softening of the unconfined fibers of the wood by the absorption of water did not operate to increase the height of the centering by the swelling of the wood, as was expected. The action of the wood during the period following the keying of the arch, apparently, was the continued yielding under the load, indicating that the elastic resistance was less than the imposed stress, and, as the deflection of the crown of each of the two main ribs was only $\frac{1}{8}$ in. at the time of striking the wedges, it would seem that the elastic resistance of the whole structure of the centering was just sufficient to carry so much of the weight of the arch ring as would deflect the arch $\frac{1}{8}$ in.

The centering was designed by the contractors. Its general arrangement and details, with stresses and sizes, are shown on Plate XLIII. All bents were supported on concrete piers built on the rock, with the exception of four intermediate ones, which were supported on 24-in. I-beam stringers spanning the creek bays. The lower story of the high bents was made of steel beams and channels braced with angles.

The remainder was of yellow pine timber. The working stress in the 10 by 10-in. posts, which had up to 13 diameters of unsupported length, was about 1 000 lb. per sq. in. As the sills, blocking, and wedges, were all 10 in. wide, the area of side-grain bearing was the same as the end grain, and the unit stress, therefore, was the same. The resulting crushing of the fibers of the yellow pine sills where they bore on the 10-in. oak wedges clearly showed that the stress was beyond the proper working load for side-grain bearing. The extreme fiber stress in bending on joists and on caps of bents was 1 200 lb. per sq. in., and ample strength and stiffness in these members was evident. The steel I-beam stringers were stressed at 16 000 lb. per sq. in., and the resulting deflection, therefore, was normal, and had no effect on the concrete work. The steel I-beam posts of the lower story had a stress of from 5 700 to 8 500 lb. per sq. in., according to the unsupported length between points of attachment of the diagonal bracing. The 6-in. steel rollers under the high bents were loaded, during the erection of the arch, to 3 400 lb. per lin. in., assuming that the rollers were all equally loaded, which is hardly possible. The total weight of the falsework on the rollers of the middle bent was 100 tons.

A peculiar feature of the structure was the method of connecting the curved joists. There were twenty-five lines of these, lapping at each bent and pinned together there with 3-in. round oak treenails. Instead of being notched to bear vertically on the caps of the bents, the joists were left straight and smooth on their underside, and beveled timbers on top of the caps of the bents were dressed to fit under them and receive their pressure. This construction produced horizontal as well as vertical forces to be resisted. In the lower bents the lateral thrust was greater than the vertical because of the steep inclination, and it was resisted by horizontal struts or rangers extending clear across. At the higher bents, where the inclination was flatter, the horizontal effect of the joist bearing was taken up by plank chocks spiked to the joists above and against the beveled timbers.

The designer assumed that the pinning of the joists would cause them to act as an arch and thus carry a considerable portion of the weight of the concrete ribs. It was apparent, however, that they did not act in this way, and also that the lagging did not arch. The length of arc was about 280 ft. and the shrinkage of the lagging in drying out was more than sufficient to accommodate the shortening



BRIDGE
ON LINE OF WALNUT LANE
OVER THE
WISSAHICKON CREEK
WORKING DETAILS OF BRACKETS AND PIERS

of the arc in the lowering of the crown by the compression of the centering, and, although the joists were very carefully bored for the treenail pins, the play in the holes and the compression of the fibers was evidently sufficient to prevent any observable arching. The placing of the joists was commenced at the crown and proceeded downward on both sides, the lower joists being fitted into the end spaces, the tendency of the joists to slip down being resisted by their linking over the crown, whereby the chain of joists was in tension before the lowering of the centering began. The proportion of rise to span is such that a change in length of arc produces almost exactly the same amount of change in the elevation of the crown. Therefore, for the deflection of $3\frac{3}{4}$ in. at the crown, the line of joists must have shortened $3\frac{3}{4}$ in., and, as the end bents depressed $\frac{1}{8}$ in. each, and as there were nineteen pin joints, the yield at each joint, by play and by compression of the wood, must have been $\frac{3}{16}$ in. When the centering was taken down, it was found that many of the treenails were permanently bent into a zigzag shape, presumably due to the effort of the linked joist to close on the pins.

It seems probable that a much stiffer and somewhat more economical structure would have resulted from fitting the ends of the joists and abutting them.

An unsatisfactory detail appeared, also, in the high blocking used between the first and second stories, and between the second and third stories. The caps and sills of the bents at these points were made double, to furnish stiffness between the posts for anticipated jacking, and 20 in. of space was then provided for the jacks. This space, therefore, had to be made up in blocking. Between the first and second stories, two 10 by 10 by 20-in. blocks were used, and between the second and third stories one 10 by 10-in. block and two oak wedges 10 in. wide and 5 in. thick at mid point. There was thus 60 in. of wood bearing across the fiber, made up of six pieces 10 in. wide. Its objectionable nature appeared in the unsymmetrical yielding of the wood under long-continued pressure, causing the pile of blocks to buckle and begin to topple. No actual failure occurred at any point, but it was evident that if the structure had been required to sustain its maximum load much longer some of the piles of blocks would have collapsed. The unsymmetrical yielding seemed to be due, at least in some cases, to an excess of sap wood on one side of the block—the

heart of the wood not being in the center of the stick. In some instances the sap wood split away from the heart wood, having swelled more than the heart wood, probably from absorption of water, and then buckled away.

It was observed that in the higher bents the caps became dished, on account of the greater compression of the interior posts. The concavity was as much as 1½ in., and the soffit of the arch rib at the crown, therefore, is necessarily that much curved, and the arch is that much thicker in the middle than at the faces. This dishing was not perceptible in the sills of the second story, and only slightly in the sills and caps between the second and top story, the transverse bracing probably acting to distribute much of the load to the outer posts.

The action of the falsework indicated that the unit stresses to which the wood was subjected, in side-grain bearing at least, were beyond its elastic resistance, and it is recommended, therefore, that the working stresses of wood in side-grain bearing, even for temporary duty, should be limited to 500 lb. per sq. in., and that the details should be made as compact as possible.

The means provided for adjusting the height of the bents, and for lowering them in striking the center, consisted of oak wedges, 10 in. wide and 20 in. long, tapering from 7 to 3 in. in thickness, placed crosswise of the sills in order to facilitate driving. When the center was lowered the wedges on the end bents at the skewbacks were loosened first by backing them out with sledges. At each of the four points they were found to have only a moderate pressure on them, proving that there was no considerable arch action on either the lagging or the joists. Then the wedges under the middle bent were slackened by backing them only enough to release a portion of the pressure, and successively all the others—working two gangs of men, one toward each end. A number of the wedges were so tightly held that they could not be started without having the weight on them partly taken up on jacks, one or two 15-ton ratchet screw-jacks being used for each tenacious wedge. One wedge refused to budge, even then, and it had to be split out with chisel bars. The evidently unequal loads carried by different posts of the same bent may be accounted for by different resisting values in the posts, and the consequent deflection and concentration of load by the caps of the bents. After all the wedges had been partially backed, the operation was repeated from the middle bent

FIG. 2.—VIEW BETWEEN RIBS, WALNUT LANE BRIDGE.

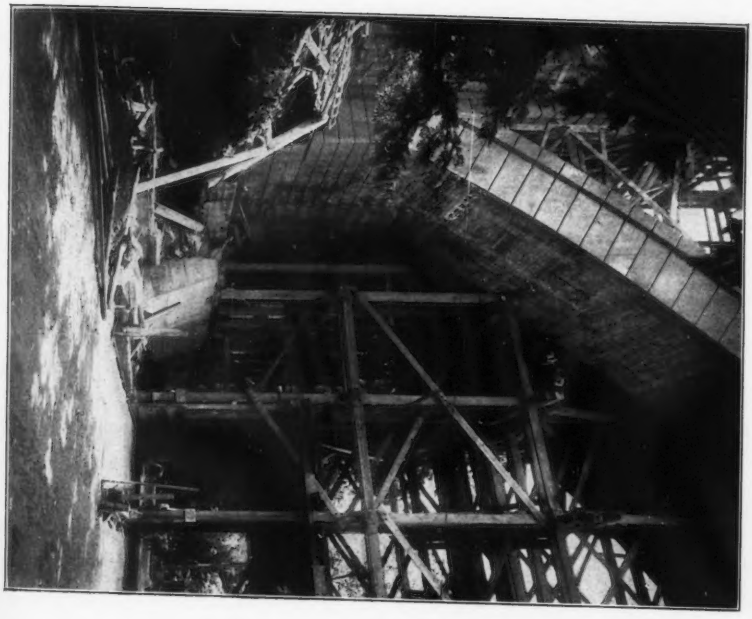


FIG. 1.—END OF CENTERING PARTLY MOVED WALNUT LANE BRIDGE.

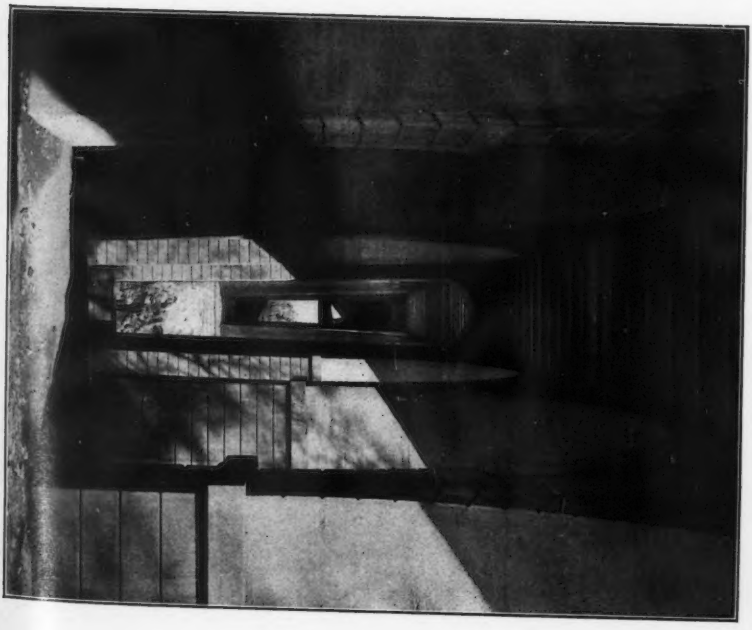


FIG. 2.—VIEW BETWEEN RIBS, WALNUT LANE BRIDGE.

to the ends, and all bents were lowered several inches clear of the arch. The longitudinal bracing of the bents acted as a truss, and carried the middle bents when their wedges were dropped, thus preventing any bents from lowering much below their immediate neighbors. Notwithstanding the apparently heavy pressure on some of the wedges, it was clear that, at the time of striking, the centering was carrying only a fraction of the weight of the concrete arch, for the deflection of the latter upon the withdrawal of the support of the centering amounted to only $\frac{1}{8}$ in. at the crown and $\frac{1}{16}$ in. at each quarter. This was the case in both ribs. The deflection was measured by three observers, each with an engineer's level, three fixed targets being used —one on the crown and one on each quarter.

One of the reasons for the economy found in the double-rib type of arch was the practicability of building one rib at a time, requiring only enough centering for one rib. Therefore the centering for the main arch was made for a single rib, while the falsework piers were made long enough for the two, and, after the first rib was swung, the whole center—with the exception of the end bents, which were moved independently—was moved over as a unit into position for the second rib. This was accomplished by using rollers under the bents, and jacks pushing on the sills (Fig. 1, Plate XXXVI). The steel bents had been erected on 6-in. solid steel rollers, and 3-in. wood rollers were inserted under the low bents. The jacks heeled against blocks chained to wooden spurs built into the piers for the purpose. The jacks were all actuated simultaneously, and a uniform speed of movement was maintained in order to prevent any skewing of the structure. This operation occupied less than three days.

The centering for the 53-ft. arches was an **A**-frame truss with a collar tie at mid height, but no tie at the heels; the thrust, therefore, was sustained by the piers. Computation of the dead weight resistance of the piers to overturning indicated that in two cases it was scarcely equal to the overturning moment, and that prudence would dictate the precaution of ties at the heels of the ribs. They were not used, however, and the construction was accomplished without any visible effect of thrust upon the piers. The dowelling of the courses of the concrete masonry of the piers, which was the result of finishing every course with the embedded stone projecting above the surface, is

doubtless to be credited with increasing the resisting moment of the piers by giving tensile value to the joints between the courses.

The vertical support of the trusses of the 53-ft. arch centering was obtained on the 6-in. projection of the coping, supplemented with a wooden trestle bent from the ground up in front of each of the low piers, and, in the case of the high piers, a timber corbel bolted to the face of the concrete. The bolts for the purpose of supporting the form timbers, wherever used throughout the work, were $\frac{3}{4}$ in. in diameter, and built into the concrete; and the nuts, 4 in. square and $\frac{5}{8}$ in. thick, were embedded from 6 to 8 in. As a rule, they were loosened slightly by unscrewing them one turn within a day or two after placing, in order to facilitate their ultimate removal, which was generally accomplished with little trouble. A few, however, were found to be gripped so tightly by the concrete that they were twisted off in the effort to unscrew them from the nuts, and had to be cut off with a chisel. This was done 1 in. or more below the surface, in order to permit a sufficient body of pointing to prevent the metal from rusting and staining the face. All holes left by these bolts were plugged with granolithic mortar and washed to match the face work.

The centers for the 20-ft. spandrel arches were built up and the lagging was nailed on before placing them in position, each center being handled and set as a unit. These were supported on the projecting caps of the piers.

The centering for the floor arches was suspended from the embedded steel beams with $\frac{3}{4}$-in. loop hangers or saddles over the tops of the beams, blocked with wood high enough to facilitate the removal of the block and the pulling out of the loop after the concrete had become self-sustaining. The centering was removed as early as was deemed safe, in order to get the hangers out with the least difficulty—under favorable conditions for setting, it was done within three days. A few of the hangers were held by the concrete so tightly that they could not be drawn, and were permitted to remain. The removal of the jack-arch centering was accomplished with a light scaffold suspended by single ropes through small holes cored in the floor concrete at suitable intervals. These holes, as well as those left by the withdrawal of the hangers, were not plugged, but were left open for the permanent drainage of any water which might get through the paving, but the spaces left by the removal of the wood blocks were filled with concrete.

FIG. 1.—FLOOR SYSTEM, WALNUT LANE BRIDGE.

FIG. 2.—DECK VIEW, WALNUT LANE BRIDGE.

All material was handled by a cableway of 800 ft. span, supplemented at the piers and on the floor by swinging boom derricks. The concrete mixing plant was under the cable.

The provisions for temperature changes throughout the structure have thus far proved to be effective in preventing cracking or spalling. The system adopted consisted in general of dividing the structure into sections, all the joints being locked with groove and tongue, and cushioned with a layer of soft asbestos felt. Steel reinforcement was not used anywhere to resist temperature effects. It has been observed in concrete structural work that unequal expansion of parts has caused rupture or spalling. Some parts—such as the rail of a parapet—may have less mass of body, and be also more completely exposed to the atmosphere, than other parts—such as the covered arch ring—therefore responding more quickly to changes in atmospheric temperature. They will then expand before the other parts, and something must yield. It has been found that, at points where such action is likely to develop, a sheet of soft mineral-wool felt, $\frac{1}{16}$ or $\frac{1}{8}$ in. thick, according to circumstances, will afford an elastic cushion which, by compressing, absorbs expansion, and, by swelling, follows contraction of the concrete, thus keeping out dirt and minimizing the appearance of the opening. Care must be taken, of course, to have the felt complete over the whole abutting surface of the joint. If any portion, particularly an edge of the joint surface, has the concrete in contact it may spall in expanding.

The floor structure, being of jack-arches, required staying at the joints, in order to take up the thrust of the arches. This was done with tie-rods. It required also double floor-beams at each joint in the middle aisle, independent of each other and separately encased in concrete. The joints are located so that in the side aisles they are over the cross-walls. Any movement in the floor structure at its joints necessarily involves a little sliding on bearings. Wherever this action is likely to occur, the seat of the sliding scale was made with a sheet of zinc, $\frac{1}{16}$ in. thick, flanged up at the back to prevent its dislodgment. Its smooth surface facilitates the sliding and it does not rust. The same device was used at the base of such of the balustrade posts or piers as occur at floor joints. The joint was necessarily made on the mid-line of the supporting bracket and wall, but it was not desirable to split the pier, so the half of the pier which had to be built over

the sliding side was bedded on sheet zinc, and the joint at the side of the pier in the balustrade completed the provision for movement.

The joints in spandrel walls were made about 24 ft. apart over the main arch—one alongside each cross-wall—and in the 53-ft. arches, because of the greater depth and consequent rigidity of the walls, the joints are only about 18 ft. apart. In the floor the joints are as much as 60 ft. apart, and the movement has not been enough at any one of them to develop a crack in the paving of the roadway, which is continuous over the joints. The joints in the sidewalks have the felt cushion.

Provision was also made for hinge action in the spandrel arches, due to the change of relative elevation of their springing points with the rise or fall of the crown of the main arch. The change of elevation of the crown from variation in temperature is $1\frac{1}{8}$ in. The spandrel piers, therefore, move up and down, each an amount proportionate to its distance from the middle of the span. Thus, the one nearest the end rises approximately one-fifth of $1\frac{1}{8}$ in., the next one two-fifths of $1\frac{1}{8}$ in., and the third three-fifths, etc. As this movement is necessarily vertical, the effect on the small arches is a bending with rotation about their springing points. To minimize the tendency of this action to spall off the concrete of the intrados at the spring, or to open cracks higher up, a sheet of asbestos felt was built in the whole width of the rib, 3 ft. 6 in., extending in from the soffit 9 in. Of course, this reduces the end bearing and increases the clear span of these arches, but they are of more than ample size for their duty.

Steel-rod reinforcement was used in general only at points where it was thought that the concrete was likely to crack on account of shrinkage or other indeterminate stresses. At the few points where load conditions produce tensile stresses, the amount of tension is so low that the flat embedded stone would furnish ample value to carry them, several tensile tests of the stone used showing strengths of from 1 000 to 1 200 lb. per sq. in. These points are the sidewalk brackets which are opposite cross-walls where steel beams were not used in the side aisles of the floor, and the 4 ft. 9-in. overhanging floor slab which is supported on the sidewalk brackets. In the former, two 1-in. square rods were embedded, extending into the concrete of the middle aisle for transverse tie purposes, and in the floor slab four 1-in. square rods were embedded in a longitudinal direction. In addition to these, trans-

FIG. 1.—BOTH RIBS, WALNUT LANE BRIDGE, AFTER REMOVAL OF CENTERING.

FIG. 2.—VIEW OF WALNUT LANE BRIDGE FROM THE CREEK.

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verse rods, ½ in. square and 18 in. apart, were embedded in the sidewalks to prevent the possible tendency to crack longitudinally at the point of overhang—the outside line of the spandrel wall. For similar reasons, rods were embedded transversely in the spandrel piers on the main arch, where the cross-web walls are much less in thickness than the pilasters which they connect; also, in the high hollow shafts of the main piers with sharp angles in section and overturning stresses, also in the abutment wings, and in the long and slender hand-rails of the balustrades.

The likelihood of concrete becoming detached from large flat surfaces of steel was provided against by the use of steel anchorage at the few points where such action would be possible. These were the projecting ends of the 15-in. floor-beams where they form sidewalk brackets, and the outer edge of the 20-in. beams at expansion joints, and the bottom flanges of all beams. The encasing concrete is clamped to the beam webs by ½-in. bent bars passing through holes in the webs, and the flange concrete is tied up with $\frac{3}{16}$-in. wire loops 3 in. apart, connected, and therefore spaced, with side wires welded to them. The concrete of the sidewalk brackets was supplied in addition with ¼-in. square steel loops tying it to the sidewalk slab. The aim throughout the design was to counteract every conceivable tendency of the concrete to open cracks from any cause, and the monumental character of the structure demanded that every precaution be taken to ensure the utmost quality of durability in every part of it.

It is probable that no completed structure of considerable size, simple or elaborate in design, has been regarded by even its author as perfect. Doubtless every progressive designer studies each structure, as it is being built and after it is finished, remembering his reasons for certain features and passing critical judgment upon their merit, and determining how defects may be corrected, or the cost reduced, or the appearance improved. As more is learned from failures than from successes, the most interesting comments on anything may be criticisms of it.

As architectural appearance—beauty of form and grace of proportion—is largely a matter of taste, the author of a work of art is hardly a competent critic of it, and, therefore, suggestions for improvement of the artistic quality of the design of the Walnut Lane Bridge must be left for others. Such modifications of structural details, for economi-

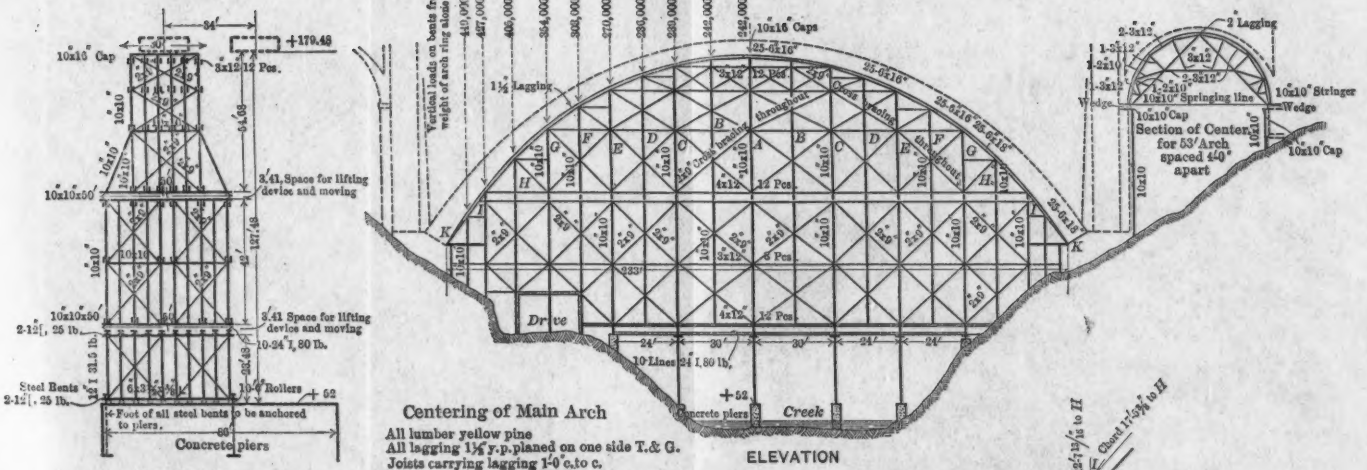
cal or other reasons, as have appeared desirable may be of interest to the Profession.

The most persistent enemy of masonry work—stone, brick, or concrete—is water when it finds entrance to it. The provision made in this bridge for drainage has not proved adequate for the complete exclusion of water from the joints, and this—a common failing in such structures—is apparently the only feature of this bridge which can be regarded as a structural defect. The points where it appears, however, are few in number, and the consequences are no more serious than slight blemishes in the appearance. Although the driveway is asphalted and has not a discernible crack in it, some water seems to get through it, and it finds outlet below where it shows on the nearest surface, and produces efflorescence. The joints in sidewalks and curbs are necessarily uncovered, and, consequently, some water finds its way through them, spreads along the floor seat ledges, and appears in stains on the finished faces of some of the abutments and piers. It is intended, in future work of similar character, to construct grooves in the seats and drain them into vertical conductors or down-spouts to be built into the wall joints and below them through the piers or abutments to some available point of discharge at or below the surface of the ground. In other bridges, efforts to prevent the infiltration of water into such joints, by the membrane method of covering the joint with web and elastic coating, have not been entirely successful, but it is believed that thorough drainage of the joints will prevent seepage to the front.

Some economy might have been attained, without lessening the stability or the durability of the work, by omitting the 18-in. concrete sheeting between the ribs of the 53-ft. arches—making four ribs, each 4 ft. wide, instead of two barrels, 18 ft. wide—and by making the spandrel walls 2 ft. thick instead of 2 ft. 6 in. Less steel reinforcement in the piers would have been justified with the high grade of concrete secured by the embedment of the flat stone, which by its complete bonding and great tensile strength resists shrinkage and other rupturing stress.

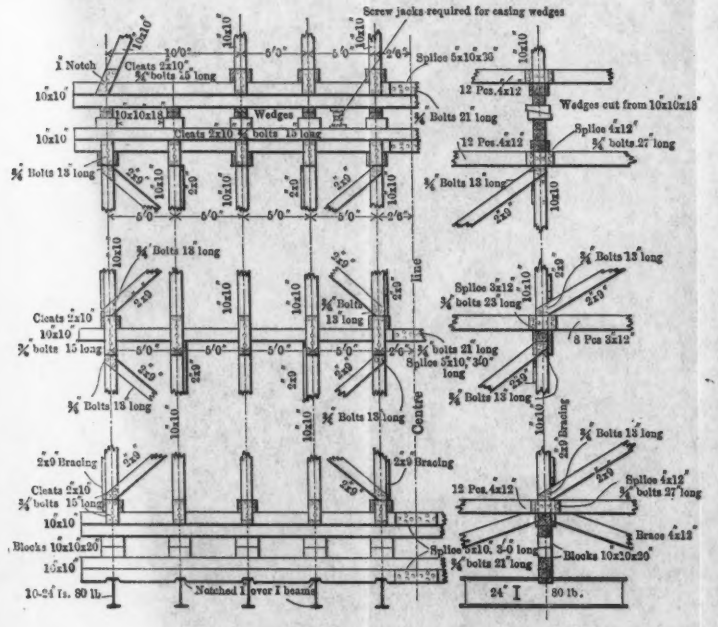
These are the only modifications of the design which have suggested themselves during the two years of construction of the bridge, which was a period of close observation and test of merit of the design.

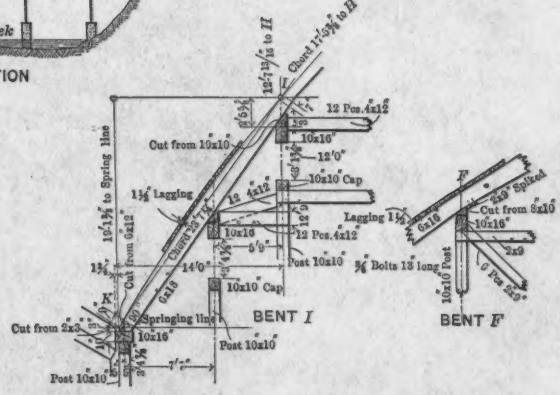
The class of concrete adopted and used, together with the character



Centering of Main Arch

All lumber yellow pine
All lagging 1½" y.p. planed on one side T. & G.
Joists carrying lagging 1'-0" c. to c.
Protection over drive 2" planks

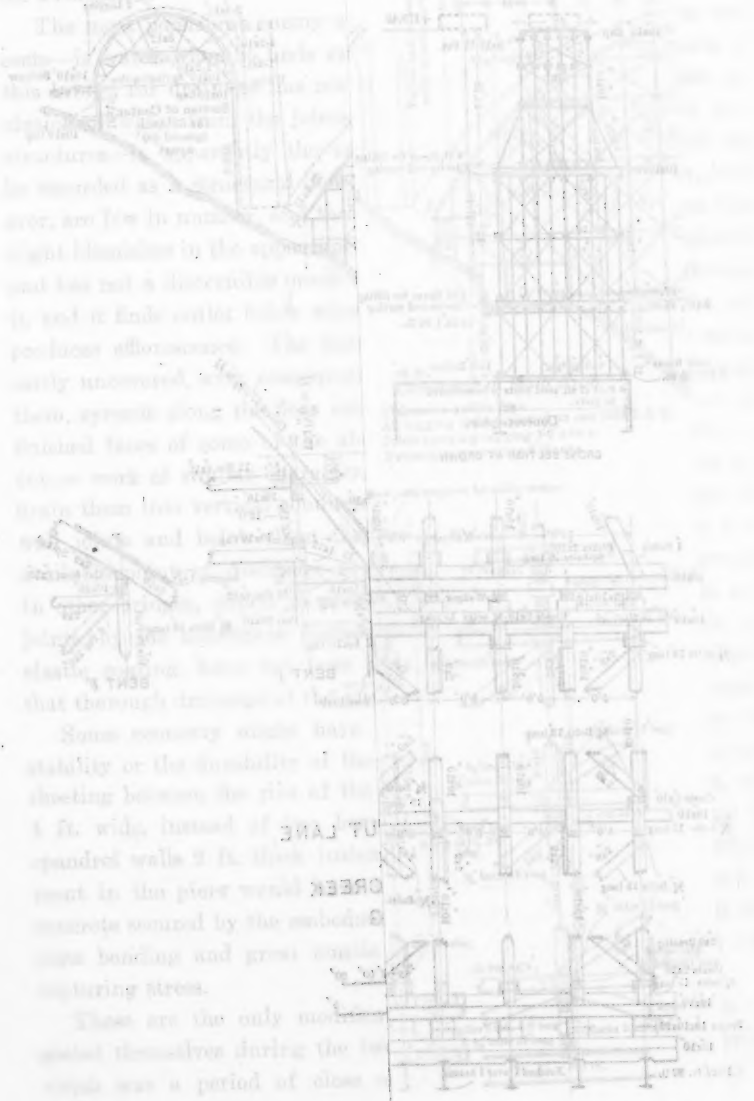




BRIDGE
ON LINE OF WALNUT LANE,
OVER THE
WISSAHICKON CREEK
CENTERING

Scales. Elevation and Section. 0 5' 10' 20'
Details. 0 1' 5'

UNIV OF MICH.



of the surface finish, and also the provision for expansion and contraction, represent the established practice of the City of Philadelphia in its bridge work, and, therefore, were not experimental. The method of constructing the main arch, as prescribed in detail in the specifications, was new, but the results fully justified it as scientific and practical, and it has been used since on similar though lighter work with equally satisfactory effect.

The total cost of the bridge, including electrical conduits and lamp standards, bush-hammering, and all extra work, was $267 000, which gives a rate of $7.60 per sq. ft. of floor surface and $0.088 per cu. ft. of space—area of profile by width of bridge.

The prices in the proposal and forming a part of the contract were as follows:

The work covered by the plans—lump sum	$253 550.91
Bush-hammering soffit of main arch ribs—lump sum	1 800.00
Additional bush-hammering—per square yard.	0.16
Additional concrete in foundations—per cubic yard	7.50
Additional concrete above foundations—per cubic yard	9.00
Additional steel—per pound	0.04

The bronze lamp standards at the ends of the bridge, and the trimmings and lamps on the iron pipe poles in the refuge bays—contracted for separately—cost $7 982. These poles were made high enough and strong enough to carry diagonal catenaries for the support of overhead trolley wires in the event of street railways crossing the bridge in the future, the purpose being to avoid all use of poles along the curbs. The total cost of bush-hammering the soffits of the main arch, approach arches, and spandrel arches was $2 147.

The total quantity of concrete of all kinds in the bridge is 19 080 cu. yd., of which 2 300 cu. yd. are in foundations and 3 100 cu. yd. in the two ribs of the main arch. The structural steel in the floor amounts to 138 tons, and the steel reinforcing rods in foundations, wings, walls, and floor amount to 54 tons.

The centering of the main arch required 885 cu. yd. of concrete, 334 000 ft. B. M. of lumber, 116 tons of structural steel, and 16 tons of bolts, washers, and nails.

The work of construction extended over two years—from July, 1906, to July, 1908—concrete work having been suspended for three months during each of the two winters. The bridge was dedicated with appropriate public ceremonies and opened to travel on December 16th, 1908, after the grading of the Walnut Lane approaches was completed.

Honorable mention is due to the contractors, Messrs. Reilly and Riddle, for the creditable manner in which they executed the work, and for their resourcefulness and energy in its prosecution.

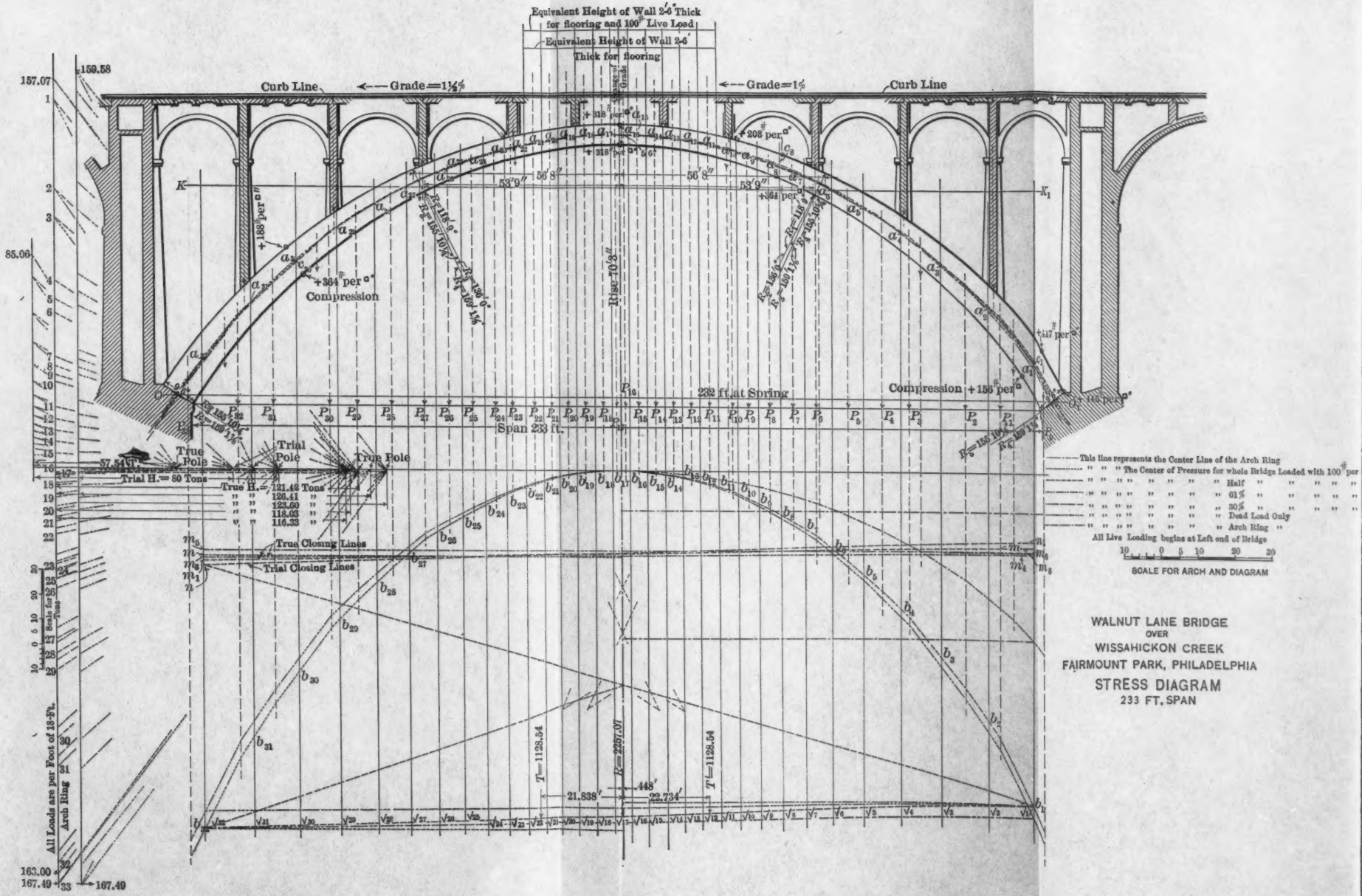
Extracts from Specifications.

The work covered by Item 1 of the proposal includes all the work necessary to make the bridge complete as per plans, and consists of excavation for foundations, grading at Piers 3 and 4, construction of all concrete and reinforced concrete work, with necessary centering and forms, floorwork, paving, iron pipe conduits for electrical wires, drainage appurtenances, manholes, electric light poles, and filling behind abutments as far as the lines shown on the general plan.

The work covered by Item 2 of the proposal may or may not be ordered, and consists of bush-hammering the under surface or soffit of the two ribs of the main arch, with pneumatic bush-hammers, sufficient to obliterate all impressions of the lagging.

In all foundations, stones of derrick size may be embedded, and smaller stones may be embedded with them, but they shall not be closer than 2 in. to any face, and they shall be used only by depositing concrete first and joggling the stones down into it. The embedded stones in the foundations of Piers 3 and 4 shall be of flat shape, and, as far as practicable, inclined so as to be normal to the direction of the resultant thrust on the pier.

The concrete of the main arch ring shall be composed of 1 part cement, 2 parts coarse sand or gravel, and 5 parts crushed stone, with embedded flat stones. The crushed stone shall be from ¼-in. to ¾-in. in size and thoroughly mixed. The embedded stones shall be not less than one-man size, and of such shape that their thickness will be approximately one-quarter of the average dimension, and shall be of quality and character equal to the best of Conshohocken stone, and be approved as such. They shall be placed in radial planes as close together as practicable, always edgewise to the centering, and embedded by first placing a sufficient depth of concrete and then working the stones into it. At the points of bearing of the spandrel piers and of the spandrel walls a number of these stones shall project above the arch ring to form a footing and bond. The concrete of the 20-ft. spandrel arches and the 53-ft. approach arches shall be of the same composition, with similar stones, of the same character, and of suitably proportioned size, similarly embedded.



WALNUT LANE BRIDGE
OVER
WISSAHICKON CREEK
FAIRMOUNT PARK, PHILADELPHIA
STRESS DIAGRAM
233 FT. SPAN

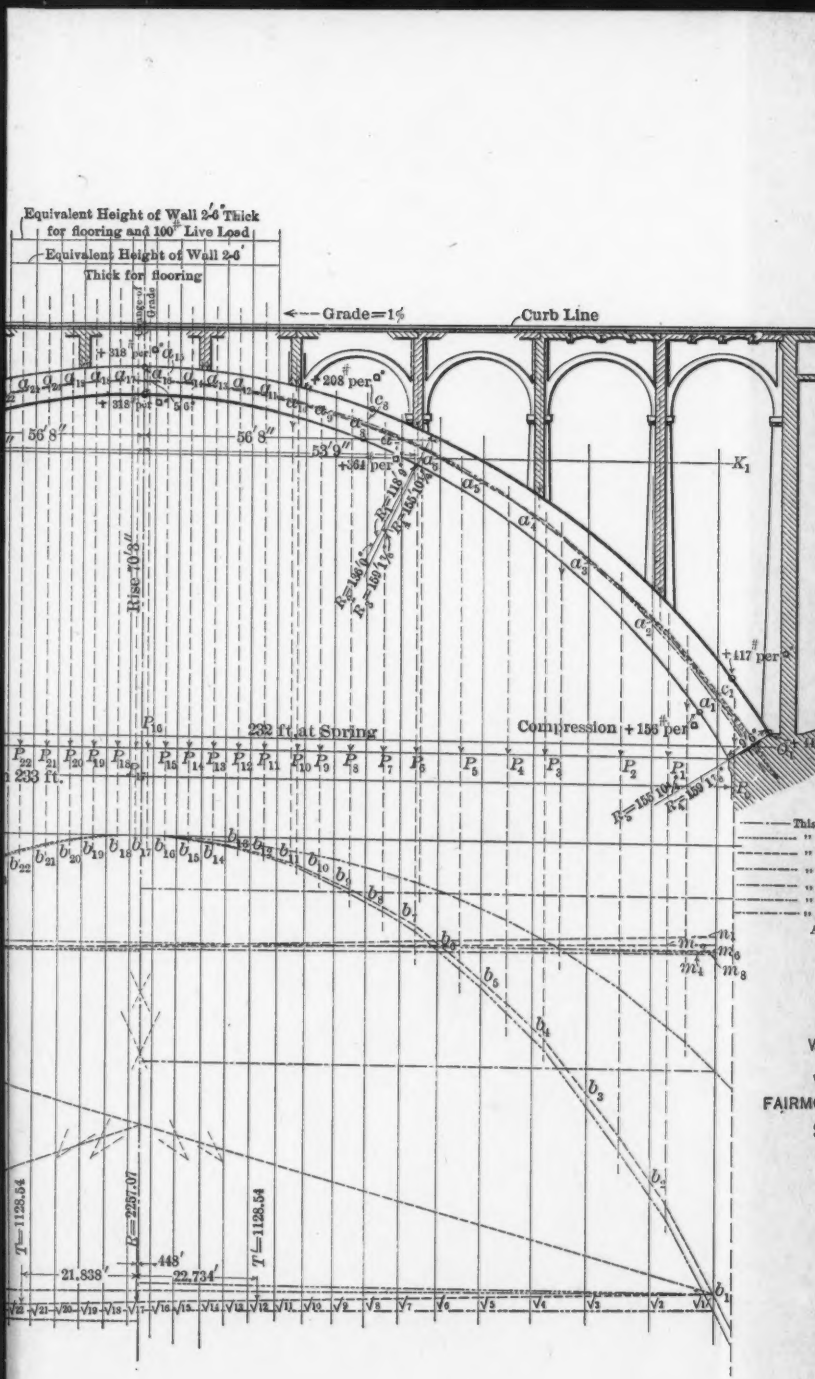
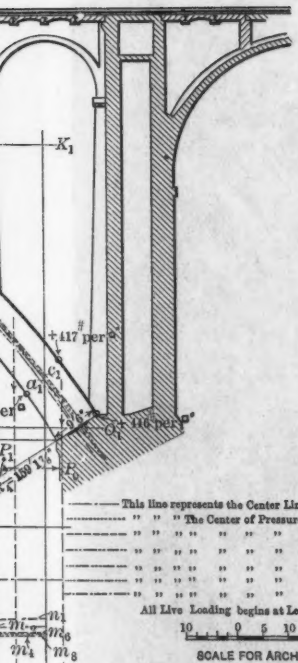

Equivalent Height of Wall 2-6 Thick for flooring and 100# Live Load
Equivalent Height of Wall 2-6' Thick for flooring
Grade=1%
Curb Line
Change of Grade
+318# per □'
+208# per □'
+364# per □'
+417# per □'
56'8"
56'8"
53'9"
Rise 10'3"
232 ft. at Spring
233 ft.
Compression +155# per □'
R=158'0"
R=149'10"
R=155'10"
R=139'1"
T=1128.54
T=1128.54
R=2287.07
448'
21.838'
22.734'

PLATE XLIV.
PAPERS, AM. SOC. C. E.
AUGUST, 1909.
WEBSTER AND QUIMBY ON
WALNUT LANE BRIDGE.



——— This line represents the Center Line of the Arch Ring
---------- " " " The Center of Pressure for whole Bridge Loaded with 100# per square foot
---------- " " " " " " " " Half " " " " " " " "
---------- " " " " " " " " 61% " " " " " " " "
---------- " " " " " " " " 30% " " " " " " " "
---------- " " " " " " " " Dead Load Only
---------- " " " " " " " " Arch Ring "

All Live Loading begins at Left end of Bridge

10 0 5 10 20 30

SCALE FOR ARCH AND DIAGRAM

WALNUT LANE BRIDGE
OVER
WISSAHICKON CREEK
FAIRMOUNT PARK, PHILADELPHIA
STRESS DIAGRAM
233 FT. SPAN

UNIV. OF MICH.

All vertical walls of neat work shall have embedded in the concrete approved hard stones, not less than one-man size, so that they approach not closer than 2 in. to any surface, and so that every course or layer will have a number of the stones projecting up from it to dowel or bond with the succeeding course.

Piers 3 and 4 must be complete to and including the skewbacks at least one month before the concrete of the main arch is deposited.

Each main arch ring shall be constructed in transverse sections in pairs—opposite sides simultaneously—to ensure symmetrical loading on the centering. These sections shall be of such length that each pair can be constructed in daylight of one day. A detailed plan showing the sections and the required order of construction will be furnished by the City. The sections shall be separated by spaces of about 2 ft. 6 in., which will constitute the keys of the arch. The keys shall be constructed not earlier than two weeks after the last pair of sections has been placed. The lower three key spaces on each side shall have on the mid-line three concrete piers constructed at the same time as the sections, to maintain the separation of the sections and prevent possible sliding on the lagging. These piers will be surrounded and built in by the placing of the concrete of the key. The joints between keys and sections shall be locked by tongue and groove 1 in. deep.

The design of the centering to be used shall be submitted for approval before proceeding with its construction. The falsework for the arches shall rest on rock or on masonry piers built on the rock. The design shall be such as to avoid, as far as practicable, bearing across the fiber of the wood, confining the pressure as far as possible to the end fiber, in order to reduce to a minimum the yielding of the structure as it receives the load. Means shall be provided for lowering the supports evenly and gradually under absolute control.

Expansion joints in the walls and all joints in the balustrade shall be locked by tongue and groove 2 in. deep, and have $\frac{1}{8}$ in. thickness of asbestos felt separating the surfaces. The balustrade joints occurring at floor joints shall have $\frac{1}{4}$ in. of asbestos felt and have the locking tongue only slightly beveled to permit free movement.

At the expansion joints in the floor, where the movement involves sliding of the floor slabs on the walls, the bearing of the sliding portion shall consist of a strip of $\frac{1}{16}$ in. thick zinc, of the width of the bearing, flanged up 1 in. at the back edge.

At the expansion joints in the sidewalks the electrical conduit pipes shall be jointed by wrapping each with 4 by $\frac{1}{32}$-in. zinc to permit movement.

The connection between the 15-in. beams and the 20-in. beams of the floor must be made with tapered, turned bolts. The connecting angle is to be riveted to the web of the 15-in. beam with four $\frac{7}{8}$-in. rivets, and the holes in the flange of the 20-in. beam are to be drilled in position

and reamed tapered to fit the bolts, which shall be not less than $\frac{7}{8}$ in. in diameter at the point of shear.

There shall be embedded in the concrete around the bottom flange of each floor-beam $\frac{3}{16}$-in. wire loops 3 in. apart, connected on each side of the beam above the flange by two longitudinal wires $\frac{1}{8}$ in. in diameter, and 3 in. apart, to hold the loops in position. There shall be no longitudinal connection below the flange. The longitudinal wires shall be secured to the loops either by welding, as in the "Clinton Wire Cloth," or by tightly wrapping, as in the "Wight Lock Woven Steel Fabric."

The vertical drainage pipes from the grate top inlets shall be 10 in. in diameter, 40 lb. per foot, asphaltum-dipped, wrought-iron or steel pipe. The drainage pipes through the foundations of Piers 3 and 4 and underground shall be 12-in. and 16-in. cast-iron pipe. The latter shall be laid with 3 ft. of earth over it, and shall extend to the rocks at the water's edge of the creek.

The electric light poles shall be complete, except lamps and wires, which are not included in this contract. The poles and fittings shall be ornamental, and the design shall be submitted and be subject to approval before erection. They shall be of such height that the lamps will be 21 ft. above the sidewalks. They shall be anchored and built into the concrete work as it proceeds. The poles over Piers 3 and 4 shall be of sufficient strength to serve as trolley poles.

The steel edging for the granolithic curb shall be furnished in lengths to match the joints in the sidewalks.

The whole surface of the floor concrete between the curbs, and also the buried portion of the face of the curbs, shall be thoroughly coated with No. 4 coal-tar distillate to a least thickness of $\frac{1}{4}$ in.

Bush-hammering of surfaces, if required to be done, shall be with pneumatic bush-hammers, and shall be sufficient to remove the surface film, exposing the grit, and to remove the impressions of joints of the lagging.

The contractor shall furnish, at points to be designated, in the arch centering, wooden tell-tale pendants with corresponding stakes in the ground to facilitate accurate observation of deflection as the work proceeds.

Where anchorage in the concrete is required to maintain the forms in true position, it shall be designed so that any projecting metal work can be removed to a depth of 3 in. from the face without damaging the face, and the cavities thus created shall be filled with granolithic mixture.

The grit in all granolithic face work, except the balustrade, shall be of dark hard stone from an approved quarry, and that in the balustrade shall be of crushed granite.

FIG. 1.—WEST APPROACH SPANS, WALNUT LANE BRIDGE.

FIG. 2.—EAST APPROACH SPANS, WALNUT LANE BRIDGE.

The cinder filling in the middle section of floor shall be selected material, and shall be thoroughly compacted by wetting and tamping.

To economize in centering and forms, the bridge superstructure may be constructed in halves—one side throughout the whole length at a time—but the two abutments and the foundations of all piers must each be monolithic.

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AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

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A REVIEW OF CHICAGO PAVING PRACTICE.

By P. E. Green, Assoc. M. Am. Soc. C. E.

To be Presented October 6th, 1909.

The matter of pavements in Chicago is in some respects a recent one, as compared with some eastern cities. Articles on New York 100 years ago, state that that city was tolerably well paved; probably Philadelphia and Boston were also; certainly, Albany was, at the time of Fulton's first trip with his steamboat. Sixty years ago, however, Chicago had no pavements except plank and graveled roads, though in 1860 it was a city of 100 000 inhabitants. The condition of the streets can easily be imagined. There are photographs in existence showing that evidently every lot owner established his own grade for his sidewalk; and Clark St. had an appearance which can only be matched in the smallest towns of the West. In 1855 two or three squares were paved with limestone blocks, and at about the same time some cobblestone pavement was laid.

The first real pavement was the Nicolson block, a pine block similar to the present creosoted block, but untreated. This was laid on Fifth Ave., between Lake St. and South Water St., in 1856. The blocks were laid with 1-in. joints filled with gravel and coal-tar, the foundation being 2-in. plank on sand. The life of this

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

pavement was short, and it did not stay in favor very long, though a considerable yardage of it was laid before the Chicago fire in 1871. Up to 1865 about 25 miles of pavements had been laid, mostly pine block and gravel macadam.

After the fire, the grade of the whole down-town and near residence districts was raised by from 2 to 11 ft., the débris of the fire being used to make the filling. The writer, when making excavations for sewers and water pipes, during the last two or three years, has frequently encountered these old Nicolson blocks, still in a good state of preservation, after being buried in ashes, brick, etc., for nearly forty years.

It must be remembered that, until the grade was raised, a large part of the city was practically a swamp. In the Forties and Fifties the citizens frequently discussed the desirability of stone block pavements such as used in the East. It was the opinion then that it would not pay to spend much money on a pavement which was sure to sink out of sight in a year or two. It was this condition which brought about the use of plank roads. They were very serviceable for many years, and were active competitors of the steam railways of those days. The writer has read an article, written for one of the Chicago papers during the Fifties, which vigorously assailed the policy of abandoning the building of these roads, and went on to prove by quotation of freight rates that freight could be moved much more cheaply by team on such roads than by railway; that passengers could be transported just as rapidly and with less danger; and that, if the city were to spend on these plank roads the amount of money it proposed to give a certain railway, dividends of 30% could be earned.

In 1875 the city began to lay cylindrical cedar blocks, and for nearly twenty years this pavement was used more than any other. The blocks were 6 in. in depth, and, like the Nicolson blocks, were generally laid on a plank foundation, though a few streets had a macadam base. A great deal of this pavement still remains, but it is in very bad condition. When first laid, it had a nice, even surface, which was very attractive, and hundreds of miles of it were put down. About 1890 it began to fall into disfavor, and from that time its use declined, as its defects became glaringly apparent. Its first cost was one thing which made it popular, the average price per square yard being from $0.85 to $1.25.

In 1882 the first asphalt was laid, and during the same year the first granite blocks. The cost of these pavements was very high, and the use of asphalt spread slowly, but, from absolute necessity, the use of granite blocks spread very rapidly, and much of this pavement, laid from 1882 to 1886, is still in service. No care was taken to dress the blocks; they were only roughly squared, and were laid with wide joints. On account of these defects, the pavement was very noisy, but it was lasting. Brick was first laid about 1891. A brick pavement on Lake Ave., from 35th to 37th Sts., was laid in 1893, and is still in use, though badly worn. The bricks were not "repressed," but were vitrified. The foundation consisted of a course of brick laid flat. This pavement has given good satisfaction under heavy traffic.

One of the drawbacks to the rapid construction of good pavements in Chicago is the law under which such improvements are made, taken in connection with the financial condition of the city. Under the Illinois law, payment is made on the basis of the benefit derived by the adjoining property. In other words, whether or not a street has ever been paved, the adjoining property owners must pay for the entire improvement, unless the County Court, which confirms the assessments for the improvement, assesses a general benefit against the city. As the city has little money, and the bonding power is very narrowly limited, the policy is for the city to have the ordinance rescinded and the improvement abandoned if the assessment against it is more than it can pay, and so it works out that the adjoining property owners pay for at least 90% of all pavements laid in Chicago. The city does not pay for intersections. The various street railway companies pave their own right of way, a strip 16 ft. wide for double track. The whole arrangement is unjust, is a hardship on property owners, and should be changed. In an indirect way, it often injures property, as streets which should be repaved are allowed to remain for years, because property owners do not feel that they will be benefited by the improvement.

All work in Chicago which is to be paid for by direct assessment against abutting property owners is under the supervision of the Board of Local Improvements, composed of five citizens appointed by the Mayor. As the work done by this Board costs several million dollars every year, a considerable engineering force is required; but, as the finances of the city are in bad shape, rigid economy is necessary all

the time. Hence, under the present system, the engineering force is totally inadequate to do the work as it should be done. As in other public corporations, there is much duplication of work in the different departments of the Board of Local Improvements as now constituted. Undoubtedly, if this organization were to be remodeled, the number of men available in the Engineering Bureau would not have to be increased in order to do the work in a much more satisfactory way, but it is difficult to make changes in the method of doing public work. This statement is given as a reason for the methods which the engineers have adopted in order to do a great deal of work with a very small force. These methods are far from ideal, but the writer thinks that they obtain the best results, under the unfavorable circumstances.

In an ordinarily busy year, some three hundred streets and alleys are improved. As it is not possible for the small force of engineers assigned to this work to set stakes constantly for the curbing, grading, concreting, etc., much of this work must be done by the inspectors, who are not engineers. Line and grade are marked on permanent objects along the street, such as sidewalks, electric light poles, etc., and from these the inspectors and foremen must work. Ordinarily, some member of the engineering force visits each job every day, but he can rarely give more than an hour of his time to each piece of work, unless some particular point requires his attention.

In order that the inspectors and foremen may follow the engineer's instructions, each is furnished by the division engineer with what is called a "profile." This is not a profile, in the ordinary sense of the word, but is a plan of the work, with many explanatory notes. These notes show where the line is marked, the offset, the height of the grade marks from the top of the curb, the width of the roadway, the roadways of intersecting streets, the depth of the gutter below the curb at summits and inlets, the location of new catch-basins, etc. The inspectors and foremen are taught how to stake out the ordinary crown with Tees, after the curb is set. The curb, of course, is always put in before the concrete foundation, and hence is a guide to the men.

Of course, careless inspectors and foremen make mistakes occasionally, and there is trouble when the engineer makes his examination, but, as a whole, the system has worked better than could have been expected. Inlets and summits are sometimes reversed, crowns are too high or too flat, but nothing better can be expected until the

engineers in charge are given more men. At present an engineering force of about twenty men, with a salary list of about $33 000, handles $4 000 000 worth of work per year, scattered over the entire city. Fig. 1 is the "Profile" of a street paved by the writer in 1908. It will be noticed that this is not a detailed, accurate plan, but is really a memorandum for the guidance of the inspector and foreman.

Before discussing the different modern types of street pavements, various features, common to all in a greater or less degree, will be taken up. These may be classified under the headings: Width of Roadway; Crown or Form of Surface of the Roadway; and Drainage.

Width of Roadway.

The width of roadway is a question which causes the engineer more grief than almost any other. Property owners seem to have a general desire for a wide roadway. Many do not know, or more probably do not think, that the wider the roadway the more it will cost. Property owners, after receiving an assessment notice showing that the roadway will be reduced from the original width, have protested strongly, seeming to think that they were being defrauded of part of their rights, but when it was explained to them that their assessment would be much larger if the roadway were wider, they have been satisfied.

Chicago has a general ordinance for the width of roadways on streets of standard width. This, however, governs only the intersections of streets which have never been paved. When a street is to be paved, the ordinance ordering the improvement states the width of the roadway. The general ordinance specifies the following widths:

Width of street, in feet.	Width of sidewalks, in feet.	Width of roadway, in feet.
100	20	60
80	16	48
66	14	38
60	12	32
50	10	30
40	6	28
30	4	22

Probably any practicing engineer will agree that, for the average street, these widths are too great, and especially for very wide streets. Many residence streets have a width of 100 ft. between property lines,

Profile of Addison St., from W. line of Clark St. to W. line of Hermitage Ave.
Roadway 44 ft. Grade marks 2 ft. High. Crown 7 and 10.
Line marked on south walk 13 ft. offset (from face of curb)

Intersecting Rdys. Racine Rdy. 38′ offset South 14.0′ Maple Sq. Rdy.—offset North 17.5′ See Engineer.
Herndon " " See Engineer Ward " 38′ " 14′
Southport " 46′ offset 16′ Janssen " 34′ " 16′.
Perry " 34 " 16′ Bosworth " 34′ " 16′.
Ashland " 38 " 14′ Marshfield " " See Engineer.
Hermitage " "See Engineer.

⊠ New Catch Basins.
○ Old Catch Basins.
× Summits.
Grades.

FIG. 1.

and on such streets a 60-ft. pavement is the height of absurdity; it is expensive to lay and maintain, and certainly cannot vie in attractiveness with, say, a roadway of 38 ft., a grass space of 24 ft., and a sidewalk space of 6 ft., leaving 1 ft. between the walk and the property lines.

This tendency to make very wide roadways is more pronounced in small towns, which have not had many paved streets, than in the larger cities, which have had more experience. A certain town, having 3 000 inhabitants, proposed to pave its main street with a 60-ft. roadway, and then changed it to a 66-ft. roadway. An assessment for such a width nearly swamps the adjoining property owners. In another city, of 25 000 inhabitants, the main street has a paved roadway about 80 ft. wide; it looks like a deserted street on the busiest day, and actually gives the stranger a bad idea of the town.

If a street is wholly residential, and say 66 ft. wide, which is almost standard, a 30-ft. roadway is ample. This will allow two teams opposite each other to be backed up to the curb, and give a good passageway between, if the wagons are properly cranked. At street intersections, there is also ample room for vehicles of the fire department to swing around the corner. This is a vital point, and is sometimes overlooked by engineers, who go to the other extreme and advocate roadways which are too narrow. A long ladder-truck will have considerable trouble and delay in turning a corner on a narrow intersection, say 20 ft. There are generally electric light poles, lamp posts, etc., which may catch the ends of the ladders as they swing around. This objection, of course, can be largely overcome by building a wide sweeping intersection, but even that is not the only consideration. A very narrow roadway sometimes assumes too much of the appearance of an alley.

For an ordinary business street, not having car tracks, 38 ft. is a good width. A great many business streets in Chicago are of that width, and it is found to be ample.

If a street has a double line of car tracks, a wider roadway is necessary. The writer would then suggest a minimum width of 42 ft., but 48 ft. would be better. The reasons for this are obvious when it is considered that the ordinary street-railway right of way is 16 ft.

A congested district, such as "The Loop," in Chicago, furnishes a problem all by itself. In such a case the roadway can hardly be too

wide. But the sidewalk space is also congested, and must be taken into consideration. The roadways in "The Loop" are generally 48 ft. wide; part of State St. is 60 ft. wide. These are too narrow, but so are the sidewalks, and it is not possible to make much change.

Crown, or Form, of Surface of Roadway.

Apparently, there is little room for variation in the crown of the roadway. What is wanted is a shape which will drain rapidly and easily, and at the same time be as nearly flat as possible in order that it will not interfere with the freighting of heavy loads. For this purpose it is natural to use the circle or some similar curve. Many quite elaborate formulas have been suggested by various engineers as a solution of this problem. These generally take into consideration the width of the roadway, the grade of the street, and the desirability of having the maximum fall near the gutter. What most of them do not take into consideration is the fact that, in a flat territory like Chicago (and in some parts of nearly every city), it is absolutely necessary to form an artificial grade in the gutter. If the minimum depth below the top of the curb is 4 in., and the maximum is 10 in., it is obvious that a surface curve which depends entirely on the width of the roadway for its form, and, at the same time, establishes the center of the roadway at any fixed elevation, will not do. In the Chicago ordinances the form of the roadway is generally the arc of a circle. In construction work, however, it has been found by experience that this does not give the best results, and the parabola is almost always used. The formula is $y = \frac{b\,x^2}{a^2}$, in which b is the depth of the gutter below the grade of the center of the roadway, a is the half roadway, x is the horizontal distance from the center of the roadway, and y is the vertical distance below the grade.

This gives a crown which is not as flat at the center, and, at the same time, drops rapidly to the gutter in the last quarter; but, for most ordinary roadways, there is no practical difference between the two curves. It is also very simple to stake it out, and is easily taught to foremen, which is extremely important. The necessary stakes for the crown are generally set at the center and quarter points. In this curve the quarter points are always one-fourth of the total drop. This is very easily remembered, and the foremen are soon able to set the stakes accurately with wooden Tees. For very wide roadways

of, say, 100 ft., it causes too rapid a drop toward the gutter, and a curve intermediate between the parabola and a straight line must be used.

In regard to a crown of this form, however, it should be stated that it is intended for pavements having a rigid wearing surface which does not grind into dust. In other words, it does not apply to ordinary macadam, because it gives too flat a center, forming a perfectly flat, or even a concave, surface which is impossible to drain. The writer considers that the nearer the section of a macadam roadway conforms to straight lines, dropping both ways from the center, the better the surface is likely to wear.

One of the important points to decide is how high the crown should be, or, in other words, how much the pavement should drop from the center to the gutter. It sometimes happens that a combination of circumstances, such as established curb grades and street railway tracks, necessitates a very low crown. The writer has known cases, where, on account of these conditions, combined with the requirements of drainage, the drop was less than 2 in. from the rails to the gutter. This was at a summit in the gutter. This is bad practice, however, and should be avoided when possible. For ordinary purposes for smooth pavements, on streets having a roadway of from 28 to 38 ft., 6 in. should be the minimum, and, for a macadam pavement, 8 in.; on wider streets, the drop must be greater.

The late Andrew Rosewater, M. Am. Soc. C. E., when City Engineer of Omaha, used, for the height of the crown, the following formula, which has met with a great deal of favor:

$$H = \frac{W(100-4P)}{5\,000},$$

in which W is the width of the roadway, and P is the percentage of grade.

This is a good rule for park roads or boulevards, or for any street having a stiff grade, when little crown is necessary, but it makes the crown too flat for level, or nearly level, streets, where drainage has to be secured by artificial grades, and where the streets are not constantly cleaned. In general, however, it is very good. It is practically the same as the 2% rule mentioned below.

One of the best general rules known to the writer is that used by the engineer of the West Park Board of Chicago, who makes the crown

2% of the roadway. This works very well on residence streets, but not at all well where there are street-railway tracks and the grade is nearly flat, as the tracks cannot follow the changes of the gutter grade, and these control the crown grade.

The Drainage.

The third general point to be considered—the drainage—is probably the most important. The whole life of practically every pavement depends largely on this. All pavements in Chicago are designed to take care of the drainage, and if any pavement is defective in that regard it will be a failure, even if it can stand the heaviest traffic. When the writer speaks of drainage, he does not mean entirely the sub-surface drainage, though that is very important. In a large city, as a general rule, there is not much trouble with the sub-drainage, because the city is thoroughly sewered, and the opening of cellars and other excavations has drained the surface and lowered the general ground-water level. Of course, a naturally sandy soil needs little drainage, but a clay soil must be watched. If it is normally in a wet condition, some provision for its drainage should be made, such as laying tile pipe across the roadway and connecting them to the sewer inlets. In this paper, however, the writer will consider only the surface drainage.

If the surface of the town is rolling, and naturally drains well, there will be little difficulty in keeping the pavements dry; but if it is very flat, like Chicago, drainage is an important problem. As has been intimated, drainage is effected by what may be called an artificial grade in the gutters. This is very readily secured if a combination concrete curb and gutter is built, in which case the gutters can be built first directly to the desired grade, and the pavement can then be built to follow it. But when a curb only is built, great care must be taken to conform to the grade desired, as the curb itself will give no indication of the proper place to locate a summit. The gutters should drain directly to a sewer inlet, and the inlets should not be more than 300 ft. apart—200 ft. would be better. If it is assumed that they are 200 ft. apart and the grade is level, the summit would be in the center, and a run of 100 ft. would be made each way. If the gutter is 4 in. below the top of the curb at the summit, and a drop of 6 in. in 100 ft. is desired for proper drainage, the inlet would be

10 in. below the top of the curb. This is as deep as desirable; if made deeper, people are likely to stumble over the curb, and the crown is made so stiff on the ordinary street that it is difficult to pull a heavily loaded wagon into the center of the street. It might be claimed that the drop of 6 in. in 100 ft. is more than necessary, but any practicing engineer who has had experience with city streets will agree that it is not too much. It must be remembered that in the ordinary residence street, which is generally not cleaned oftener than once in two or three weeks, but is flushed nearly every day, a large part of the street sweepings accumulates in the gutters. Added to this is the sidewalk dirt, which is also swept into the gutter. In park roads and in streets which are swept and flushed every day, the rate of fall may be reduced, and good results will be obtained with a fall of 3 in. in 100 ft. This will reduce the sewer inlets by one-half; but the street cleaning must be of the best.

Another point to be noted is the drainage of the intersections. Ordinarily, this is a source of great trouble, especially if the intersecting streets are not at right angles. In this case the result of the crown intersections is a large flat space. The greatest care must then be taken in staking the grade for the concrete or other foundation in order to make it conform as nearly as possible to the theoretical intersections of the curves. Wherever possible, it is the writer's custom to raise the center of the intersection an inch or two higher than the established grade, in order to facilitate the drainage. An ideal intersection would have three inlets at each corner, placing one back of each of the property lines and one at the curb-line intersection, and bringing the pavement cross-walks flush with the sidewalks at these corners. It is not very often that this arrangement can be followed, as it is somewhat more expensive than the ordinary way of having one inlet at the curb corner; but the result in keeping a dry intersection would soon pay for it.

There is one more general point which might be mentioned, as it applies more or less to all pavements, and that is the foundation under the wearing surface. In late years, the concrete foundation has come into use almost everywhere for streets having a heavy traffic. Its usual thickness is 6 in., which, as a general rule, is sufficient; for streets of very heavy travel 8 in. is better. In streets in which necessarily many openings have to be made concrete is undoubtedly the

best foundation, but many residence streets and even business streets having a heavy traffic are but seldom disturbed.

Under such conditions, the writer is convinced that a thoroughly rolled stone base is the equal if not the superior of any concrete base. It keeps its line and grade just as well as concrete, if not better. As the heavy roller, used in compacting the stone, works continuously back and forth on this foundation for many hours, it will have every opportunity to discover any weakness in the subgrade. It may be said that the rolling of the subgrade should disclose this in any case, but, in addition to the subgrade rolling, the broken stone gets another tremendous pounding, to make a close, tight foundation, for which the concrete depends on its cement. Of course, a perfect concrete is inferior to no foundation, but, in ordinary work it is much easier to approach a perfect macadam foundation than a perfect concrete one. The writer believes that any engineer who has had experience in making either hand-mixed or machine-mixed concrete will agree with this.

In support of this general theory (which the writer is aware is opposed by many engineers), attention is called to the down-town streets of Chicago. Practically all are on a rolled foundation such as has been described, and, although the surface is badly worn, and the problem is complicated further by the poor construction of the car tracks, these streets are serviceable; and these pavements have been in service for from 15 to 25 years. The old granite blocks are rounded and worn, and will be replaced by a more modern surface soon; innumerable openings have been made in the streets, and this is an unfavorable location for such a foundation; but it is hard to believe that any concrete anywhere can show a better record. As an example of modern rolled streets, which show up very well, Harrison St., from Western to Rockwell, might be mentioned. This is an asphalt pavement on 6 in. of macadam, and was laid in 1903. It is in the best of condition, after 6 years of heavy traffic. Desplaines St., from Madison to Van Buren, of granite blocks on 6 in. of macadam, laid nine years ago, is a model surface to-day, though the traffic is very heavy. Wentworth Ave., from 55th to 63d Sts., was laid with brick on 6 in. of macadam, in 1897. Since then a conduit has been constructed in the street, and in 1908 the car tracks were reconstructed after being in bad condition for years, and yet the pavement

is serviceable to-day and is good for 5 years more. These are only a few examples. The only modification which would seem to the writer to be beneficial would be to put in an 8-in., instead of a 6-in., foundation on such streets.

In many places in which concrete has been used it would have been much cheaper to use a stone or gravel macadam. The writer knows of one small town in which the engineer provided a concrete base for a brick pavement, in spite of the fact that the street had been graveled for twenty years or more, and there was an abundance of good gravel in the vicinity.

In many cases a concrete foundation is a necessity, as, for example, in streets of very heavy traffic, streets having a bad foundation, such as wet clay, etc., and streets in which, undoubtedly, many openings will be made from time to time. As the concrete is generally from 6 to 8 in. thick, and is spread over a large surface, and not deposited in bulk, as in walls, abutments, etc., it is necessary to take more than ordinary care to obtain good concrete. A small sand pocket may make little difference in a large mass of concrete, but in a sheet like a pavement it may be the cause of a large hole which will ruin the wearing surface and extend with great rapidity under traffic. It is a vital necessity, therefore, to have inspectors at the delivery point and at the mixer, if one is used. If the concrete is mixed by hand, every care should be taken to see that it is thoroughly done, that too much water is not used in the mortar, causing the cement to be washed away, or too little, making a porous concrete.

The machine-mixing of concrete is one of the serious problems in street work. With many types of mixers, it is necessary, for a large part of the time, to have the mixer some distance from the work, such as at the next street intersection, etc. This means that each batch as mixed must be transported to the work in carts, wheel-barrows, etc. This is where the trouble comes in; the constituents of the concrete separate; in bottom-dumping carts, much of the "slush" or mortar, leaks out on the way; the heavier stone is shaken toward the bottom; and the net result is that the concrete is exceedingly uneven in quality when delivered. This trouble is largely eliminated by using a mixer placed close to the work and delivering the concrete on a board from which it can be shoveled into place. There are many places, however, in which it is very difficult, and sometimes impossible, to use such a

machine, and recourse must be had to hand-mixing or the distant machine. The writer would always prefer hand-mixing.

The proportions of the concrete are 1:3:6, using Portland cement, sand or limestone screenings, and broken stone, gravel, or crushed slag. The sand is standard torpedo. If limestone screenings are used, instead of sand, they must be clean, sharp, and free from dust or large flat pieces. There is no apparent difference in concretes made with sand or screenings.

One of the troubles in concrete foundations, caused by poor cement, poor mixing, etc., is the transverse cracks which appear later. These are especially bad for an asphaltic top, and cracks in this wearing surface are often caused by this fault and not by lack of bitumen, but longitudinal joints or cracks in the concrete appear to affect the asphaltic surface but little.

During the past two years, practically all the car tracks in Chicago have been reconstructed. This involved slight changes in grade in many cases, and often, on an asphalt street, the pavement was completely cut out half way back to the gutter. In laying the new concrete no particular effort was made to obtain a good bond with the old, other than by brooming a little and spraying it with water. It is probable that in most cases there is no real bond between the old and new concrete. After the concrete had set (generally in 4 or 5 days) the new top was laid. Some of this construction has now been down for 2 years, and no signs of cracking or failure have been seen.

The Wearing Surface.

In this paper no attempt is made to discuss macadam pavements of various types, as this subject has been treated very thoroughly in a recent paper by A. H. Blanchard, M. Am. Soc. C. E.* Strictly speaking, macadam is not suitable for a city pavement, but only for parks and light residence streets.

Brick Paving.—Ordinary vitrified brick is one of the most popular paving materials in small cities, but is not as popular in the larger ones. It requires a good foundation—as does any pavement—and the only problems in connection with it are the quality of the brick and the kind of filler. All block pavements require a sand cushion or bed of 1 or 2 in. under the blocks.

* *Transactions*, Am. Soc. C. E., Vol. LXI, p. 445.

In regard to the foundation, various substitutes, cheaper than a macadam or concrete base, have been used, but not with any great success. In some small towns having a sandy subgrade the bricks are laid directly on the sand after rolling the subgrade, but the pavement soon wears into ruts under any large amount of traffic. A base course of bricks instead of concrete has also been used as a foundation, but, if laid properly, this is as expensive as concrete, or more so, and is not as good.

The quality of the brick is determined by water absorption, specific gravity, and abrasion tests. To these might properly be added a test of the uniformity of shape. It happens frequently that bricks which are otherwise good have been warped so much by the firing that they have to be rejected on account of their shape. If a uniformity test were applied, and the provision were made that not more than a certain percentage of the bricks should depart from the required shape, the pavement would undoubtedly be improved. The form of this provision might be as follows:

"Not more than 5% of the bricks shall depart a greater distance than ¼ in. from the standard block furnished by the manufacturer. A greater percentage than this shall be cause for the rejection of the entire lot from which the samples tested were taken."

It is not possible to give exact sizes for paving brick, as those of different manufacturers vary slightly, but it can be specified that they shall not be smaller than a certain size, say 3 by 4 by 8 in., or larger than 3½ by 4 by 10 in., and this is generally done.

The water absorption test is very important, as it measures the porosity of the brick. If the weight of water absorbed is more than a very small percentage of the weight of the brick, the latter is almost sure to go to pieces in two or three years under the freezing and thawing action of winter. The Chicago specifications limit the quantity of water absorbed in 72 hours to 3% of the weight of the brick, and some of the engineers desire to reduce this to 2 per cent. This would probably be done if it were not for the fact that so many bricks are rejected under the 3% clause; for it is thought that at the 2% rate no brick at all could be obtained.

The specific gravity required is generally about 2.1. The various manufacturers have no trouble in meeting this specification, and it has been found by experience that this gives a good heavy brick.

The abrasion test is also very important. A brick may be dense and heavy, but so brittle that it will not stand pounding. For this test the rattler is used. It consists of a steel cylinder, about 28 in. in diameter and 20 in. long. The sides are composed of steel staves having open spaces between to permit the escape of broken fragments. The size of the charge is about 15% of the volume of the rattler. Inside the rattler a number of pieces of loose heavy castings are placed. The rattler makes 30 rev. per min., and is run for an hour; bricks which lose more than 20% of their weight are rejected.

Disregarding the quality of the brick, the point over which there is most controversy is probably the filler. Three fillers are in general use: sand, bitumen, and cement grout. Sand is the oldest and cheapest form, but is not now used to a great extent. The objections to it are that it does not make the joints water-tight, and that flushing the street forces the sand out of the joints and loosens the brick.

The most popular material is either a cement grout or a bituminous filler such as coal-tar, pitch, or asphalt. The writer believes that, as far as wear is concerned, there is little to choose between tar, asphalt, and cement grout fillers. In favor of the cement grout, it is urged that it makes the pavement approach a monolithic whole, protects the edges of the bricks, and is most easily cleaned and kept clean. Its chief objections are that it increases the noisiness of the pavement, and that after grouting, the pavement must be undisturbed for several days in order to set thoroughly. In any street of large traffic, it is nearly impossible to comply with this last requirement, and a corps of police would be needed to keep the traffic off the apparently finished street. For this reason alone, the authorities in Chicago have entirely abandoned grout, and now use a bituminous filler exclusively.

The grout is generally made in the proportions of 1 part of cement to 1 or 2 parts of sand. It is generally maintained by engineers that it must be carefully mixed in a suitable box, and that every precaution must be taken to prevent the separation of the sand and cement. From his experience, the writer believes that this idea is good, but its practical attainment is difficult and unsatisfactory in any form of box. Formerly, it was his custom to insist on the box, but there was so much trouble that one of the contractors was finally permitted to try his own scheme. He made a thorough mixture of the dry sand and cement and swept it into the joints, thus filling them about half

way to the top. A spray of water was then applied and the grout was formed in the joints. Plenty of water was used, care being taken that it was not forced directly into the joints so as to wash out the mixture.

For a second course, sand and cement were mixed dry in piles directly on the pavement, and then the hose was turned on it. By using squeegees and the hose, this was thoroughly mixed, and the whole was then spread over the pavement and into the joints. The results have been of the best. During the past two years, there has been occasion to examine several openings made in pavements grouted in this way, and there is yet to be found a bad joint. In one particular case, where the adjoining street was afterward paved with asphalt, and the brick pavement at its intersection was exposed and, for about half the width of the brick, undermined, the grout joint held it up in spite of the fact that the material wagons were constantly driving over this intersection.

The advantages of a bituminous filler are that it tends to make the pavement less noisy, and also makes any repair easier than when grout is used, for it is very difficult to make openings in the latter if it has been properly applied.

The bituminous filler is either a pitch, which is the result of the distillation of coal-tar; or some asphaltic compound. The name pitch simply means coal-tar from which the water and lighter oils have been driven off by distillation, the product being hard at ordinary temperatures. If the distillation is carried too far, parts of the heavy oils are also driven off, and the result is a hard, dense product, which cracks very easily, and is not suitable for the purpose.

The pitch filler is either poured on the pavement from buckets and squeegeed into the joints, or else poured directly into the joints with a special bucket. The latter method is a little more expensive, but is the better, as the pitch on top of the brick is a nuisance.

The asphaltic filler is probably better than the pitch filler, as it does not crack so much in cold weather, nor does it lose its oils as rapidly as the tar products. In other words, a filler having an asphaltic base is more stable than one having a coal-tar base; but, as a general rule, it is also about twice as expensive. It is always applied with a special pouring bucket, and is never squeegeed.

If a grout filler is used, some provision should be made for expansion. Generally speaking, an expansion joint of ½ in. along the gutter,

and ½ in. longitudinally every 50 ft., will be ample. If there are street-railway tracks, it is advisable to lay the bricks for about three courses longitudinally with the street, and use a bituminous filler, not on account of expansion, but because of the incessant vibration caused by the cars, which tends to break the bond of the grout.

Secretary Blair, of the National Paving Brick Manufacturers Association, recommends the total elimination of the transverse expansion joint in brick paving, maintaining that the longitudinal joint is sufficient, but the writer is not prepared to agree with his reasoning.

Each of the various companies promoting tar, asphalt, and cement grout fillers insists that its product is the best. During the past three years the writer has made openings in many brick pavements in which a pitch filler had been used, and has nearly always found the joints well filled, even when the pavement was many years old. The same could no doubt be said of the asphaltic filler, but it costs more than twice as much as the pitch, and it has not been proved that it will really give any better service, though from its nature it should be somewhat superior to pitch.

Although cement grout at first makes a smoother surface, in the course of three or four years the grout in the joints is broken by the blows of countless horse-shoes, and after that there is practically no difference between its appearance and that of a tar-filled street. The joints then rapidly wear down, and become wide and rough.

An example of exceedingly good service given by brick may be seen on La Salle St., from Randolph to Madison Sts. This pavement has been in service for 14 years, and it is still usable, though nearly worn out. The traffic on this street is enormous, and innumerable openings have been made.

The principal objections to a brick pavement are its noisiness on residence streets and the wear of the edges on heavy traffic streets. Its advantages are that it is easily cleaned, is not affected by the weather, and, under proper conditions, has a very long life.

Stone Block.—The next grade of pavement, considered purely from a wearing standpoint, is the stone block. It is made with dressed blocks, generally about 5 in. wide, 5 in. deep and from 8 to 10 in. long. The block should be a hard stone of medium grain, which will resist the weather and not wear smooth and round. Sandstone of various kinds, granite, and what might be called "near-granite," such

as quartzite, which is really a sandstone, rhyolite, etc., are used. Various kinds of artificial blocks, such as iron slag, etc., have also been tried, but, in this part of the world, at least, have not met with much favor.

Much better stone block pavements are now laid than formerly. This is due largely, if not entirely, to more careful dressing, which makes a smoother and more regular block; and more care is taken in laying them. Formerly in Chicago the blocks were only roughly dressed, and were laid with wide joints. The result was that the edges were very much rounded under traffic. No investigation was made as to whether or not the blocks became slippery under traffic. Blocks which in the past have worn badly will not now be accepted. Samples of the so-called granite block to be used are required by the authorities when a bid is made, and the location of the quarry from which the stone is taken must be stated.

Although called granite blocks, it is not insisted that they shall be a true granite; but they must be what is commercially known as a granite, and must show a record of efficient service. The result has been, that the stone block pavements laid during the past 5 or 6 years, are very good, being nearly as smooth as brick, and of course much more durable. Examples of granite paving which the writer considers very good, may be seen on La Salle St., north of Lake St.; on North Ave., west of Clybourne Ave.; on Michigan St., east of State St.; and on many other streets. Practically no granite has been laid in "The Loop" district for many years, as it has come to be recognized that it is too noisy for an office, retail, and shopping district, and property owners are willing to pay for the renewal of a quieter pavement.

A sandstone block in the proper location has many points in its favor, but, like everything else, it shows up badly when in the wrong place. It is not suitable for streets of exceedingly heavy traffic, such as "The Loop" district. It does not get as smooth or rounded as the harder granite, but it wears rapidly, and in holes. If it wore uniformly it would be ideal, as far as service is concerned, but under Chicago's enormous traffic it fails. An example of its use may be seen on Washington St., just west of La Salle St. Kettle River sandstone was used, and it has been in service about 4 years. It has been necessary to patch it with granite. If this had been laid on a

secondary business street, such as the principal street of an outlying district, or on a street where the business is principally wholesale, it would probably have given good satisfaction for many years. Even under its present conditions, it has not worn smooth, and the joints are still close and regular.

Considering quartzite as a sandstone, though it is generally called granite, the record is much better. It has a high crushing test, nearly equal to granite; is somewhat coarser; does not wear as smooth, and cuts very nicely. Much has been laid in Chicago, with good results. Most of it came from Sioux Falls, S. Dak. Stone from about a dozen quarries has been used in Chicago, but nearly half of them have been eliminated during the last 4 years, either because the granite was too soft, or was so hard that it could not be properly dressed.

It is generally recognized that a good granite pavement will outwear all others, but it is very noisy indeed, and is being relegated to wholesale and factory districts, where the traffic is of the heaviest truck character, and noise is a secondary consideration. It is also rather unsanitary, and is expensive to keep clean.

The filler is usually of mixed gravel and tar. Grout has been tried with success, as far as the pavement itself is concerned, but has been abandoned on account of traffic conditions, it being practically impossible to keep the roadway blocked for a week in order to let the grout dry.

Wood Block.—One of the most popular wearing surfaces of the last few years is obtained with preserved wood blocks, generally called the creosoted wood block pavement. Its advantages are that it is smooth, noiseless, sanitary, and very easily repaired. About its only disadvantage is that it is slippery. It has been used for many years abroad, and has given good satisfaction. In America it has to overcome the bad name given to wood block pavements by the old untreated cedar block pavements, generally laid on plank alone without any other foundation, with results which can easily be imagined. The modern wood pavement is entirely different, the blocks being rectangular, instead of cylindrical, and always laid on a good foundation.

The cycle of operations by which the block is prepared is as follows: The seasoned or partly seasoned timber is cut into 4 by 4-in. blocks from 5 to 10 in. long. These are placed in a vacuum chamber and the air is exhausted. The wood preserving oil is then admitted

to the chamber, and forced by pressure into the wood. It is desirable that the blocks shall be evenly impregnated with the preservative all through, especially if they are to be laid on a street of moderate traffic, as they will probably rot out before wearing out.

The Chicago specifications first required that 12 lb. of oil be forced into each cubic foot of timber; later, this was changed to 16 lb., as it was found that 12 lb. did not fill the fibers. Experience has shown, however, that southern long-leaf yellow pine requires from 18 to 21 lb. of oil to impregnate the fiber thoroughly, and the question then arises whether or not the wood is injured by the pressure necessary to inject the oil. It takes 6 hours for a charge, and a pressure of about 170 lb. is required to force the oil into the wood. About the same pressure is required for a 16 or 20-lb. treatment, the latter simply taking a longer time. If a lighter pressure were used, say 100 lb., the time required for the oil to impregnate long-leaf yellow pine thoroughly would probably be from 16 to 20 hours. It might be noted here that Michigan Ave., south of Van Buren St., was laid with blocks having a 10-lb. treatment some 10 years ago, and is in perfect condition to-day. This street has a heavy boulevard traffic. Only a very few openings have been made.

It has been generally specified that southern long-leaf yellow pine alone shall be used. It is quite possible, however, that this is a mistake, and that many other varieties of pine, such as tamarack, Norway pine, etc., which are cheaper, could be used, and with excellent results. It is probably a little more difficult to season them properly, but it is claimed by the tamarack promoters that it can be just as well done, that there are fewer large knots, and that there is less likelihood of checking around the circumferences of the rings of the timber. In the new specifications, several kinds of timber, such as tamarack, Norway pine, black gum, etc., are specified.

One of the troubles with blocks having a cross-section of 4 by 4 in. is the fact that, owing to the accumulated oil on their surfaces, it is sometimes difficult to tell which is the proper way to lay them. It is obvious that they should be laid on edge, for, when laid with the grain or flat, they very soon wear. In Chicago streets, therefore, in spite of rigid inspection, some of the blocks have been placed on their sides. The way to remedy this is to use a block having a different width and depth, such as $3\frac{3}{4}$ by 4 in.

It has been stated that in New York City blocks 3½ in. deep are used. In Chicago their depth is 4 in. In contrast with these figures, the practice abroad is to use blocks having a depth of from 5 to 7 in. New York City has been criticized severely by various engineers for using such shallow blocks. In a recent report* made to the City of New York, upon paving abroad, it is stated that in Paris the blocks, when badly worn, are taken up, the worn part is cut off, and the blocks are then relaid. In spite of this, their average life is only 9 years. Chicago pavements do better, for there are good creosoted pavements which are 10 years old, and, as far as can be seen, are good for 10 years more. The probable reason for the short life in Paris is stated to be the excessive amount of sprinkling.

It is also probable that another reason is the superficial treatment the wood receives. According to American ideas, the blocks are hardly treated at all, simply being given a hot bath in the preserving oil. It would seem that French engineers now recognize that this was a mistake, for, in a paper entitled "The Future of Exotic Wood as Applied to Pavements," presented to the First International Road Congress by M. Louis Mazerolle, Engineer of Bridges and Roads, of Paris, and translated for the City of New York, it is noted with approval that wood blocks, creosoted under pressure, are still in good condition after a service of 15 years.

After reading of various French methods, the American municipal engineer will certainly come to the conclusion that American construction is decidedly better, but that the French maintenance is very much superior.

Another point is the fact that the larger the blocks the more difficult it is to impregnate them completely. Therefore, when they are sawed off, a large part of the resulting surface will certainly be of nearly untreated wood. Even if the block is turned over, it will probably begin to rot in a very short time. This course of treatment, therefore, will probably prolong the life of the pavement only 2 or 3 years, and then the process will have to be repeated. Taking into account the increased cost of the blocks, the necessarily heavier treatment they must receive to impregnate them, the fact that high pressure may injure the fibers of the timber, while if a low pressure is used, the time required to impregnate the timber will be prohibitively in-

* *Engineering News*, Jan. 21st, 1907.

creased, the cost of repairing the deep blocks by sawing and the likelihood of inadequate results, it is quite probable that American practice is better. It might be mentioned that, in the same report, New York was also criticized, by implication, for the lack of depth of its granite blocks. This is not good reasoning. It is not probable that a granite pavement ever failed because of the shallowness of its blocks. Granite wears out because of the rounded edges, becoming slippery, etc., thus making a bad footing. A deep stone will certainly not make a better pavement; what is required is a better dressed block and more care in laying.

Returning to wood blocks: It has been the custom in Chicago to lay them very tight, in order that the surface may be sealed as much as possible. There has never been any trouble until the past year, when the tightness was carried to an extreme, sledges being used to make the joints as close as possible. During the winter of 1908-09 the pavement heaved. In spite of this, it is probable that no trouble would have occurred had the timber been of the proper kind. The heaving was undoubtedly caused by water absorption and then freezing and thawing. In one case tar was used as a filler, and in the other case sand.

Experiments made by the Kettle River Quarries Company* have demonstrated that, with the treatment specified (16 lb.), there is absolutely no water absorption, even when the block is left for weeks in water. There is not much expansion in wood. Trautwine states that white pine expands 1 part in 440 000 for each degree Fahrenheit. As expansion joints are always provided when such a filler as sand or grout is used, and as none is needed with a tar filler, and considering the fact that, in each case where failure occurred, the wood was supplied by a company other than that which had previously furnished it, the conclusion is inevitable that the fault was in the treatment or the timber. Though the work was not done under the writer's supervision, he is of the opinion that the fault was in the timber, in that it was of too rapid a growth, having large cells, etc., which could not be well sealed. Undoubtedly, also, in one case at least, the blocks were laid unnecessarily tight, for while they should be tightened as much as possible with bars, they should not be pounded to refusal. This point is taken up later.

* Discussion by Walter Buehler, Assoc. M. Am. Soc. C. E., at the meeting of the Illinois Society of Engineers and Surveyors, February, 1909.

In Chicago, the Kreodone-Creosote oil, produced by the Republic Creosoting Company, has always been the standard, and the results have been satisfactory. In a paper presented before the Illinois Society of Engineers in 1908, Mr. John B. Hittell, Chief Engineer of Streets, Chicago, shows that a large percentage (about 9%) of the oil of a half block volatilized in the first three days it was exposed in the even temperature of a closed room. That more than this is lost on the street is almost certain, and the action is very evident from the appearance of the blocks piled on the street and waiting to be used. In the strong sunlight they very soon begin to check badly in all directions, and the action continues until they are laid in the pavement, filled, and covered with sand. After they are in service, there is little apparent loss.

It was early recognized that to use as a standard the treatment advocated by any one company was anything but scientific, but at that time the city had no adequate laboratories in which to try out and analyze the different processes. Since the installation of an asphalt laboratory in 1907, however, it is in better shape, and the specifications have now a more scientific basis. The principal points are as follows:

The quantity of oil used is 20 lb. per cu. ft., but it is the writer's belief that, for the reasons previously stated, this is unnecessarily large. The oil must be a pure, coal-tar product, free from any adulteration; it must not contain any petroleum, nor more than 5% of matter insoluble in benzine. Oil obtained or partly obtained from water-gas tar, or oil tar, will not be accepted.

The specific gravity of the oil shall be at least 1.1 at 25° cent.

In the matter of volatilization of the oil, which, as has been stated before, was found to be a very serious one, the following provisions have been made:

Up to 150°	cent.	Nothing must come off.
" " 210°	"	Must not volatilize more than 5 per cent.
" " 235°	"	" " " " " 15 " "
" " 315°	"	" " " " " 40 " "

It is the writer's opinion that it will be exceedingly difficult, if not impossible, to enforce rigidly these percentages, but, undoubtedly, an attempt should be made to approach this standard.

Another very important point, which has been mentioned before, is the large size of the cells and hence of the annular rings of the timber; this is also noted in the specifications, and the provision is inserted that, in long-leaf yellow pine, the number of annual rings shall be not less than seven per inch, measured radially from the center. This provision will bar out first-growth timber which has had too rapid a growth, and in which the cells are so large that it is difficult to seal them. As previously mentioned, one of the apparent failures was probably due to the freezing and thawing in the cells of the timber.

One of the troubles with creosoted block pavements has been caused by the brow on streets having car tracks. Such streets, ordinarily, have more than the average amount of traffic, as they are generally the business streets. The Chicago specifications provide that the blocks shall be placed on a sand cushion of 1 in., but even if the greatest care is taken with the concrete, it is not possible to get an absolutely uniform surface, and the sand bed will vary from ½ to 1½ in. As the blocks must be rolled after they are laid, the sand bed must be graded so that when first laid the blocks will project somewhat above the car tracks. When the street is rolled the blocks are forced down, and afterward the traffic pounds them down still farther. In the course of time, if the blocks have not been set high enough, or the sand cushion is too deep, they are pounded down until they are considerably below the edge of the rail, making an ugly bump in the street, and a depression in which water will stand along the edge of the rail. Fig. 2 shows this condition; it should be noted that the shoulder, *A*, is too high, and *B* is too low. Water collects at *C* and disintegrates the pavement.

On the other hand, if the blocks are set so high that they never pound down to the top of the rail, the pavement along the brow will fail in a few years from brooming along the edge, thus tending to tip at the rear end of the block. Fig. 3 shows this condition. Both *A* and *B* are too high, the block is ground and distorted at *C*, and the accumulation of water at *C* tends to destroy the pavement.

In Figs. 2 and 3, *C* is the controlling point, and must be higher than any point beyond it.

Either of these cases can be avoided only by the greatest care in shaping the concrete foundation at the brow and shoulder. If the blocks are 4 in. deep, and 1 in. of sand bed is used, the concrete should be 4¾ in. below the top of the rail. Sand 1¼ in. deep will compact to 1 in.

The block will then be ½ in. above the rail when first laid. Rolling and tamping will reduce this to ¼ in. and the traffic, after a shorter or longer time, will pound the brow down to the rail level.

It seems to be a custom in Chicago and elsewhere to require that the courses be laid at an angle with the line of the roadway, generally, 45 degrees. The result is an enormous amount of waste, because every course must be ended and finished by a skew block, and the regular blocks must be cut at an angle to make this skew. These skew blocks, on account of their sharp, triangular ends, are also particularly susceptible to wear, and must be and are a point of weakness. It seems to the writer to be the height of absurdity to continue this 45° or similar angle. It is simply following the custom of brick paving, without any of the same reasons. In brick paving the joints are comparatively wide, therefore it probably saves a good deal of wear on the edges to lay the bricks at such an angle, but a preserved wood pavement is as smooth as asphalt, has practically no open joints, and will certainly be cheaper and better if the courses are laid at right angles to the line of the street.

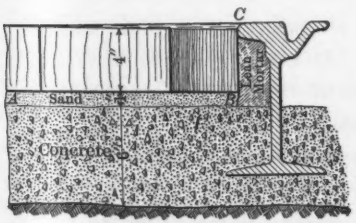


FIG. 2.

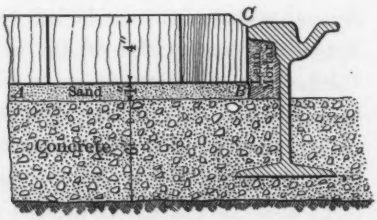


FIG. 3.

Asphalt.—Asphalt is undoubtedly the most popular pavement in the larger cities of the country to-day. Approximately 50% of all the pavements now being laid in Chicago are asphalt. The reasons, of course, are its noiselessness, cleanliness, ease of repair, good looks, and comparative cheapness. It costs less than brick, only 60% as much as creosoted block, and about 50% more than standard macadam. It is, however, probably the most technically difficult pavement to lay, and should never be laid without expert advice.

As a material, it has been abused in past years more than any other. It has been laid by irresponsible firms, without adequate advice, has been laid on streets for which it was not adapted, and the result

has been many failures. The authorities, however, have come to the understanding that it needs the best of supervision and inspection, and, under such conditions it has become nearly the ideal paving material. Formerly it was the custom to accept the guaranty as being all that was necessary to get good results. The city engineer had very little knowledge of the material, beyond that imparted by the companies doing the work. It is different now; Chicago, like other cities, has established an asphalt laboratory, and samples from each job are examined daily by a competent chemist.

The modern asphalt pavement is composed of the foundation; the binder course, consisting of small crushed stone, of 1 in. in diameter and less, mixed with asphaltic cement, intended to give body to the top; and the top or wearing surface, consisting of asphaltic cement and a filler of fine sand and Portland cement.

The specifications were formerly very general indeed, no percentage of bitumen was specified, nor was any attempt made to regulate the manner of preparing the refined asphalt, the flux (or oil to be mixed with the refined asphalt, to render it a workable cement), or the sand grading. Now all this is looked into closely. It is specified that crude natural asphalt shall not be heated to a temperature of more than 400° Fahr. in order to drive off the water. It is also specified that at least 98½% of the bitumen which is soluble in cold carbon bisulphide shall be soluble in cold carbon tetrachloride.

Mr. L. Kirshbaum, Asphalt Chemist for Chicago, shows by detailed experiments* just why this provision was inserted, as it was objected to by some of the native asphalt companies. Suffice it to say that he demonstrates that the carbenes (which is a term applied to those asphalt bitumens which are more soluble in carbon bisulphide than in carbon tetrachloride) are an evidence of lack of care in the preparation of asphalt from the native asphaltic oils, and for that reason the clause is justified.

Formerly the use of Texas or California asphalts for the wearing surface was not permitted, but of late years it has been demonstrated that if these heavy oils, having an asphaltic base are prepared properly, they are the equal of any of the Trinidad or Bermudez asphalts. Therefore the Chicago specifications now provide for the use of Cali-

* In an article in *The Municipal Engineer*, December, 1908.

fornia asphaltic petroleum, distilled at a temperature of not more than 600° Fahr., giving a residue not harder than 40° penetration at 77° Fahr., by the Dow machine. The requirement as to carbon tetrachloride, of course, applies to this asphalt in common with the others, together with a test to show that most of the light oils have been driven off in distillation. The requirements are that the sample tested shall not lose more than 5% by weight, nor shall the penetration be reduced more than 50% when heated for 7 hours at a temperature of 325° Fahr.

The fluxing material is also carefully specified, and as carefully examined for penetration, and for water, coke, and other impurities. The whole idea is to prepare an asphaltic cement which will not become too soft in summer or too brittle in winter. Equal care is taken with the sand grading; the sizes of the grains must be such that practically none shall pass a 200-mesh sieve, all shall pass a 10-mesh sieve, the bulk of it shall pass a 40-mesh sieve, while a smaller part shall pass an 80-mesh sieve.

The binder, also, is now what is called a tight material, as opposed to the old open binder. In the tight mixtures the voids are nearly filled with small stone or torpedo sand, in the open binder this stone was of nearly uniform size, about ½ in. The result of this was many so-called "fat" spots, or a concentration of the asphaltic cement, which had a tendency to rise through the wearing surface and make a weak place in the pavement, and this invariably wore into a hole. The tight binder mixture prevents this, as the asphaltic cement is distributed more uniformly through it. It also has an advantage which is immediately apparent. It may be teamed over, without breaking up under the traffic. This is necessary because the material for the top or wearing surface must be delivered in wagons over it. The old open binder very often broke up under such traffic, and then became no better than the old cushion coat used 20 years ago. It is too soon as yet to see the improvement in the wear of the pavement, as thus inspected and prepared, but the writer thinks he is safe in saying that not a single pavement laid during the past 2 years, when this care has been taken, has cracked or shown signs of failure.

Asphalt, according to experience in Chicago, is not suitable for streets having the heaviest commercial traffic, such as those in "The Loop" district, as the best pavements laid thus far have shown a

tendency to wave and wear rapidly. As an example of this, six blocks on State St., from Randolph to Jackson Blvd., may be cited. This stretch was laid by one of the most responsible companies in the business. It employs a chemist constantly to look after the mixtures. Part of the pavement was under a 10-year guaranty, and part under a 5-year guaranty. On both stretches the guaranty will expire within a year. This street has an enormously heavy traffic, being in the heart of the city's retail shopping district. A repair gang is kept busy patching almost constantly and the surface is quite wavy. On account of the character of the traffic, every effort was made to get a perfect construction, but while the pavement is kept in fairly good condition with constant repair, the surface cannot be called a success on such a street.

Again, property owners, in their enthusiasm for a nice, smooth pavement, have called for asphalt pavements on streets where the traffic is so light as to be almost nothing. Here, again, the result has been bad. Lack of traffic, apparently, is nearly as fatal as too much. The surface cracks, shrinks, and dries up, and in three or four years is a mass of unsightly cracks. Between these extremes the results have been best. Many examples of this may be shown: La Salle Ave., from Chicago Ave. to Clark St., which is 15 years old, has a fairly heavy traffic, and, although it is rather badly worn now, 80% of its surface is in good condition. Dearborn Ave., between the same limits, is 13 years old, and is in excellent condition. Indeed, one block of this is more than 25 years old, and can hardly be distinguished from the remainder. Both these streets have been repaired within the last year.

As a general rule, however, the impossibility of making repairs, except with a special plant, makes asphalt a not altogether desirable pavement for a small town, unless that town is very near a large one. The amount of work to be done in such a town is not enough to pay the contractor to keep a permanent plant there, and therefore he will make repairs but once a year, if possible, and bad places in the pavement may go several months before being repaired.

In Chicago nearly all the various brands of asphalt, both native and imported, such as Trinidad, Bermudez, California, Texas, etc., have been used, and under proper supervision they are all apparently good. For some of them too short a time has expired to be sure of results, but none has shown signs of failure as yet.

While streets may be seen in Chicago where asphalt has been in service for 20 years or more, it is probable that, after about 10 years, there is practically no "life" left in the wearing surface. Practically all the oils, both light and heavy, have been volatilized by that time, and a sample of the top may be disintegrated between the fingers, producing nothing but clean sand, and leaving absolutely no stain on the fingers. The blackness may be seen in small grains, and is probably the "coke" of the old asphaltic cement. The writer has made this experiment many times with the same result. On streets with light traffic, the pavement will probably be good for some years after that, but on streets with heavy traffic the surface goes to pieces rapidly when this stage is reached, and even constant patching will not keep it in good condition. An example of this may be seen in North Clark St., between Kinzie and Division Sts. This was paved in 1897, has a very heavy traffic, and has been repaired every year, for the past few years. In the fall of 1908, it was thoroughly repaired, but at present (April, 1909) it is again in bad shape, new holes constantly appearing.

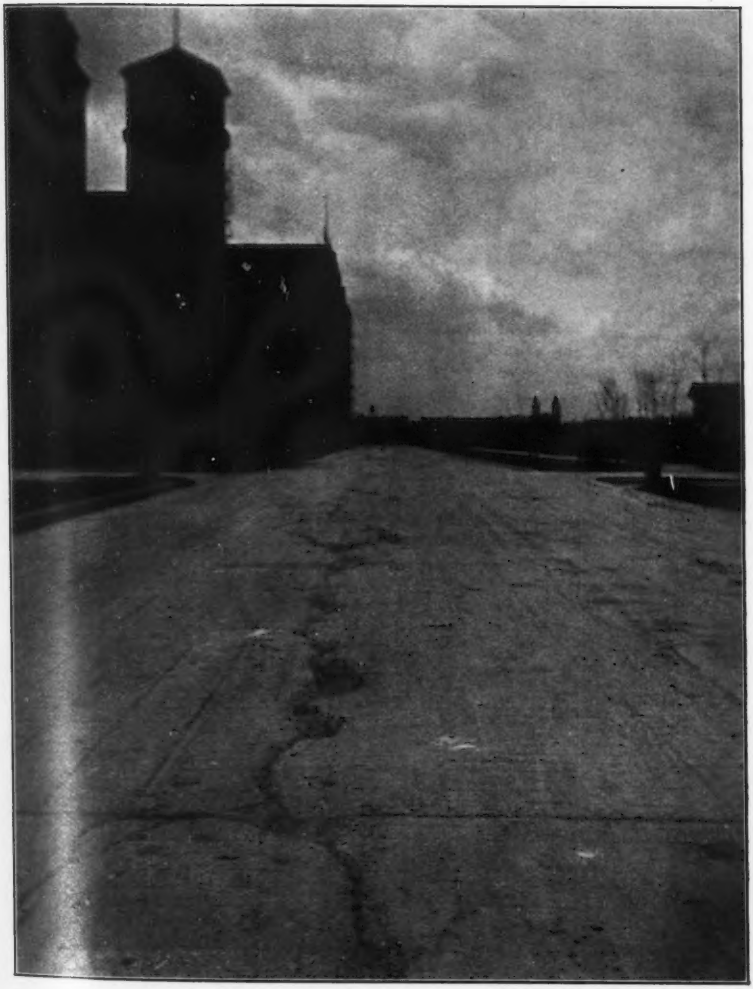
Bitulithic pavement, nearly allied to asphalt, has become popular of late years, and has much to recommend it. It is patented, and for that reason cannot be laid in Illinois by special assessment. In Chicago, however, it can be and is laid by the park boards, which do not work under the same law. It has been very successful. It consists of asphaltic bitumen mixed with a carefully-graded aggregate of crushed stone. The mixing and grading of the stone are done by special patented machinery, and, when the mixture is delivered on the street, it is spread and rolled like ordinary asphalt. On the surface is spread a flush coat of bitumen, and chips are then spread over this flush coat and rolled in. The result is a fine, even surface, not as smooth as asphalt, but in other respects very similar. On account of the fact that the city cannot legally lay such a patented pavement, none has been laid under the writer's supervision, but from observations of pavements put down by the park boards, his opinion of it is very high. One stretch of ¾ mile, on Sheridan Road from Irving Park Blvd. to Byron St., which he has had under observation, has been down 4 years, and has been very satisfactory. The street has an exceeedingly heavy boulevard traffic, but there have been practically no repairs, the surface is in excellent condition, and has not cracked, or become slippery. The cost of the bitulithic pavement is slightly greater than that of

asphalt, and, if equal precautions are taken in the selection of the bitumen, there is no reason why its life should not be as long, if not longer.

Another adaptation of this same principle—which may or may not be an infringement of bitulithic patent—is the so-called "Mineral Rubber" pavement, now being laid by the park boards in Chicago. It consists practically of run-of-crusher stone, 1½ in. and less, mixed with asphalt. No particular attempt is made to grade the stone, and the ordinary brands of asphalt may be used. A flush coat of asphalt is spread, and the top is sprinkled with torpedo sand. In Chicago it has been mixed with portable mixing machines and laid on the street in sections across the roadway. These sections are about 8 ft. wide, and are separated by wooden strips. The surface has been very good, smooth and not slippery, but is somewhat uneven, due probably to lack of rolling. The writer is opposed to the strip method, and more in favor of continuous construction. Joints are always points of weakness, and he sees no good reason for making an exception in this case.

Miscellaneous Pavements.—Various kinds of concrete pavements have been laid with some success in Chicago. These consist essentially of a bottom course of about 5 in. of ordinary 1:3:6 concrete, and a second course of richer concrete, laid before the first has set. The whole is then finished with a flush coat of rich mortar, which is smoothed off and grooved into rectangles of about 5 by 10 in. The greatest care must be taken to get proper expansion. A system of streets in the vicinity of the Sears, Roebuck Company's plant on the West Side of Chicago, only 3 years old, is very badly cracked, both longitudinally and transversely, on account of expansion. In places this action has nearly overturned the curb. Wide cracks extend down the center of every street, and the pavement is in bad shape. Plate XLVI is a photograph of one of these streets. No comment is necessary. A similar pavement, laid on Ohio St., near the Lake Shore, which is only 1 year old, but in which adequate expansion joints were provided, shows no signs of failure.

These pavements are very desirable in many ways, being easily cleaned and easily repaired. They have been used in several alleys. The surface has not worn too well, in some places, under heavy traffic. One alley in the rear of the Lexington Hotel and another at 31st St.



CRACKS IN CONCRETE PAVEMENT IN A CHICAGO STREET.

UNIV. OF MICH.

and Wabash Ave., have shown decided signs of failure, the joints being battered down and the surface disintegrating, although they have been finished for less than 1 year. They were laid in two courses, a botttom course, probably of 1:3:6 concrete, and a top course of rich mortar, probably 1:2. These are extreme examples, and the pavement should not be condemned on these alone. They were not laid under strict supervision, being done by what is called private contract.

Various types of patented pavements, of wood, concrete, etc., have been tried from time to time, but the trial pavements are too limited in area to enable one to comment intelligently on them.

TABLE 1.—MILES OF PAVEMENT OF VARIOUS KINDS LAID IN CHICAGO BY SPECIAL ASSESSMENT, FROM 1903 TO 1908, AND AVERAGE PRICES PER SQUARE YARD.

Year.	ASPHALT.		BRICK.		CREOSOTED BLOCK.		GRANITE.		MACADAM.	
	Miles.	Average price per square yard.	Miles.	Average price per square yard.	Miles.	Average price per square yard.	Miles.	Average price per square yard.	Miles.	Average price per square yard.
1903..	49	$2.46	7	$2.65	0.03	$3.25	3.5	$4.33	20	$1.38
1904..	43	2.50	5.5	2.91	0.57	3.52	4.5	4.06	31	1.38
1905..	48.7	1.985	6.6	2.26* 2.62‡	0.20	2.65†	4.0	3.60	38.9	1.22
1906..	52.4	1.88	5.1	2.38* 2.64‡	0.75	3.23	3.9	3.59	43.5	1.15
1907..	57	2.17	3.25	2.26* 2.62‡	0.90	3.78	5.5	3.93	18.9	1.21
1908..	38	2.09	10.6	2.35* 2.51‡	4.7	3.93	6.5	4.06	16.7	1.25

*All streets.
†One job, blocks laid on old macadam.
‡All alleys.

Table 1 shows the various classes of pavements laid each year by special assessment, and the average prices per square yard. Asphalt is by far the most popular in the city districts. In the outlying districts, macadam is naturally the most popular, on account of its low first cost. The prices include such grading as is necessary to prepare the subgrade, where a street is being repaved, but do not include any large amount of grading in any case. The price per square yard includes the pavement complete, that is, the foundation and the wearing surface, but not the curbs, sewer adjustments, etc. All pavements are under a 5-year guaranty except macadam, which is under a 2-year

guaranty. It will be noticed that the price of brick is very high. This is because brick has been used almost exclusively as an alley pavement in recent years. All the foundations (except macadam) are of Portland cement concrete, 6 in. in depth. The macadam consists of an average of 12½ in. of material, 9 in. being limestone in two courses and 3½ in. being granite in one course.

TABLE 2.—QUANTITIES OF PAVEMENT IN CHICAGO AT THE PRESENT TIME, 1908.

	Miles.	Approximate Yardage.
Asphalt	416	6 955 000
Asphalt blocks	2	30 000
Rock asphalt	0.6	10 000
Brick	107	1 260 000
Cedar block	452.50	8 100 000*
Concrete	3	80 000
Creosoted block	7.25	130 000
Granite	63	1 260 000
Medina stone	1.15	20 000
Macadam	552	9 000 000
Novaculite macadam	2	32 000
Slag macadam	6	96 000
Total	1612.5	26 915 000

*Practically all the cedar, some 8 000 000 sq. yd., is now worn out and useless; probably 90% of the remainder is in fairly good condition, and 10% is in bad condition.

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PAPERS AND DISCUSSIONS

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THE PURIFICATION OF THE WATER SUPPLY OF STEELTON, PENNSYLVANIA.

By James H. Fuertes, M. Am. Soc. C. E.

To be Presented October 6th, 1909.

The plant for the purification of the Steelton water departs, in several particulars, from previously constructed types, and embraces features of some novelty. The interest expressed by engineers and municipal authorities who have visited it, and seen it in operation, inclines the writer to believe that, although the plant is not a large one, a description of the conditions leading to its design, as well as an account of its construction and the results thus far obtained in its operation, would bring out an instructive and useful discussion of certain subjects on which more information is desirable.

The plant is a slow sand filter having a capacity of 3 000 000 gal. of filtered water daily; and among its novel features are:

1. The use of coarse-grained filters instead of subsiding basins in the preparatory treatment of the water;
2. The small area required—about one-fourth of that ordinarily required for a slow sand filter plant of equal capacity;
3. The system of handling and caring for the sand removed from the slow filters during scraping;

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

4. The system of determining, measuring, and applying the chemical solutions;
5. The construction of, and the filtering materials in, the roughing filters;
6. The method of operating the roughing filters, and the safeguards to prevent the deterioration of the effluent;
7. The control of the rate of filtration of the slow filters.

Among the novel features pertaining to the operation of the plant are the following:

1. The very high rate of operation of the slow filters;
2. The small quantity of coagulant necessary to secure satisfactory results—about one-fifth of that required in a mechanical filter plant of equal efficiency, and about one-eighth of that required to secure equally satisfactory results with coagulation and subsidence;
3. The ease of manipulation of the chemical dosing, and the certainty in its application;
4. The long runs of the roughing filters when no coagulant is necessary, and the consequent saving in wash-water, power, and attention;
5. The long runs of the slow filters, and the resulting large quantities of water filtered between scrapings;
6. The uniformity in the quality of the water applied to the slow filters, and consequently the satisfactory results obtained in the final purification;
7. The brilliancy of the effluent from the slow filters;
8. The absence of trouble with rusty hot water and troubles from the destruction of the solder in copper closet tanks;
9. The moderate cost of construction and operation.

The Borough of Steelton, having a population of approximately 15 000, is in Dauphin County, Pa., on the same side of the Susquehanna River as Harrisburg, and about 3 miles farther down. Topographically, the territory within the borough limits consists of a flat portion, about 20 ft. above the level of the river and from 1 000 to 1 500 ft. in width, which extends the whole length of the borough, and the residential district, which extends up the side hill to the east, and is much broken, and traversed by streets having for the

most part heavy grades. The principal street skirts the eastern edge of the flat area, the larger portion of which is occupied by the works of the Pennsylvania Steel Company.

Original Water-Works.—The original water-works plant was built by the Steelton Home Water Company, and was purchased by the borough in the latter part of 1899 for $150 069.48, since which time improvements to the intake, pumping station, and reservoir, and extensions of the street mains swelled the total cost to $223 000, on January 1st, 1906, of which $145 000 was the cost of the street mains, $38 000 the cost of the reservoir, and about $40 000 the cost of the intake and pumping station. The plant, at that time, obtained its water from the Susquehanna River through a 16-in. intake pipe reaching out approximately 1 400 ft. from the pumping station, to within about 150 ft. of the shore of Stuckers' Island. The original supply had been derived from wells dug on that island, but in a comparatively short time it became impossible to keep up the supply from the wells, and a pipe was extended about 50 ft. out into the river from the most southerly well, the water being then drawn from the river to the wells and back to the pumping station through the 16-in. pipe. Subsequently, owing to the silting up of the wells and their inaccessibility during high water in the river, a section of the 16-in pipe was taken out at a point about 150 ft. east of the island, allowing the water to enter the pipe from the river direct; it was in this condition on January 1st, 1907.

The water flowed through this broken-into intake pipe to a brick-and-concrete-lined well situated about 150 ft. from the river, just east of the Pennsylvania Railroad tracks, and was pumped from this well to the service reservoir, 270 ft. above the river, by two 1 500 000-gal. Deane, compound, direct-acting, outside-packed, plunger pumps, through a 12-in. force main. An additional 16-in. main, connected up at the pumping station to permit either pump to pump direct or to the reservoir, parallels the 12-in. force main for about 1 200 ft., extending to Second Street on Connestoga, and is cross-connected with the distribution system and the 12-in. force main. The service reservoir, when full, holds about 7 500 000 gal.

Character of Susquehanna River Water.—The Susquehanna River, above Steelton, has a water-shed of approximately 24 300 sq. miles. It drains practically one-half of the State of Pennsylvania, and about

6 080 sq. miles in the southern central portion of New York State. Upon its banks are many large cities, including Binghamton, Cooperstown, Corning, Elmira, Hornellsville, Horseheads, and Owego, in New York State, and Athens, Altoona, Bellefonte, Clearfield, Emporium, Harrisburg, Hazleton, Huntingdon, Lewistown, Lock Haven, Mahoning, Mifflintown, Mount Carmel, Nanticoke, Pittston, Plymouth, Sayre, Scranton, Shamokin, Sunbury, Towanda, Tunkhannock, Tyrone, Wilkes-Barre, and Williamsport, in Pennsylvania. Approximately, 1 000 000 persons reside in the cities and towns along the river and its branches above Steelton, of which probably 750 000 are located along the North Branch, about 100 000 on the West Branch, and 150 000 on the Juniata, which streams unite a few miles above Steelton to form the main Susquehanna River. In addition to this urban population, there is a very large rural population, and, of the larger cities, there are upwards of 60, having a total population of about 700 000, which have sewers discharging directly into the river and its branches.

The anthracite coal fields of Pennsylvania are on the water-shed of the North Branch, and along the whole length of the river and its tributaries there are many important manufacturing industries, the wastes from which are discharged into its waters. The West Branch lies largely in a sandstone country, and the principal industries along it have been in connection with the manufacture of lumber. The Juniata lies very largely in a limestone district.

In the days of canal navigation, several dams were built on each of the main streams, canals paralleling the streams on one side, or both, between the pools formed by the dams. The principal dams on the North Branch were at Clark's Ferry, Sunbury, Nanticoke, Wilkes-Barre, and Binghamton; on the West Branch, at Lewisburg, Muncy, Williamsport, Lock Haven, and Queen's Run; on the Juniata, at Millerstown, Lewistown, Newton, Hamilton, and Huntingdon; and on the branches at Pipers, Petersburg, Big Water Street, Little Water Street, Willow Street, Donnellys, Smokers, Mud, Williamsburg, Three Mile, Crooked, Frankstown, and Hollidaysburg. The existence of these dams is mentioned because they play an important part in the modification of the character of the river water during the season of low water as well as during the early stages of the fall and spring floods. Essentially, it will be seen that these various streams consist of a series

of comparatively steep natural slopes joining level pools, which, during low and moderate stages of water, form subsiding basins in which suspended matters carried by the water may be deposited in greater or lesser amounts. During freshets, however, the deposits which may have accumulated in the pools during the preceding quiet periods are scoured loose from the bottom, picked up by the rushing waters in a more or less changed condition, and carried down the stream in large quantities.

The larger part of the water-shed is semi-mountainous, the main streams flowing, for the greater part, through unerodable, and comparatively steep valleys, and the hillsides being largely denuded of forest areas; for this reason the stream and its branches are more or less flashy, big floods rising to a height of many feet in a few hours and falling almost as rapidly.

The amount and character of the sediment in a turbid water is of importance in determining the method of preliminary treatment best adapted to prepare the water for final purification. At Steelton, the quantity which will subside to the bottom of a 1-gal. bottle will sometimes reach a depth of $\frac{1}{2}$ in., and is usually a composite of sand, coal dust, leaves, stones, silt, nitrogenous matter, coagulated clays, as well as aluminum hydrate and the hydrated oxides of iron mixed with various forms of dirt, bacteria, and other matters.

In the Susquehanna River water at Steelton a brown sediment, varying from red to yellow in color, either flocculent or otherwise (the flocculence depending largely on the stage of the river and the influence of the stage on the accumulation of bacterial masses and clay particles into agglomerations), is generally characteristic of the water in the west channel, although a sediment of this color may sometimes occur over the whole width of the river, following heavy general rains. A microscopic examination of this brownish sediment shows that it consists of particles of coal and dirt agglomerated by growths of small water plants and filamentous bacteria, probably of the order *Bacterium,* or *Leptothrix.*

A greenish-black sediment, resulting from the mixture of coal dust and clay, is generally flocculent, and is characteristic of the water in the east channel during heavy floods.

A grayish sediment, due either to finely-divided suspended matter other than clay, or to the hydrates of aluminum and of iron, which

settle to the bottom when the river is low, is characteristic of the water in the east channel, and is locally called "sulphur water."

A black sediment, generally more or less flocculent, consisting of finely-divided particles of coal from the mining districts, is characteristic of the east channel, and can be observed on the bottom of the river below islands and bridge piers, and in other protected positions when the river is at low stages.

All possible gradations of color intermediate between these are observable at different times owing to the commingling of the different waters.

The presence of red or yellow clay in the Susquehanna River water at Steelton, in increasing quantities, is almost uniformly accompanied by decreasing alkalinity; such water clears up slowly by subsidence. The water carrying flocculent sediment, which is generally black during floods but often the clean hydrate when the river is low, will clarify very quickly by subsidence. Such waters are characterized by low alkalinity and correspondingly high permanent hardness.

The color of the turbidity of the Susquehanna River water, therefore, is a fairly good index of its character, and, during the operation of the filters, plays an important rôle as a guide to the proper use of the coagulant.

The changes which take place in the character of the sediment, as the result of bacterial activity, are intimately associated with the iron, sulphur, and alkaline constituents of the water. Shortly after the Harrisburg experimental filtration plant was put in operation, in the fall of 1902, one of the roughing filters, then operating without a coagulant, became suddenly clogged to a condition of water-tightness within a very few minutes after the turbid raw water was turned on, and an examination disclosed a heavy blanket of black, sticky mud on the surface of the sand. It was surmised that this mud might be similar to that which was reported to have accumulated in the sedimentation basins at Cincinnati during the experiments conducted by George W. Fuller, M. Am. Soc. C. E., but a chemical examination of the mud on the Harrisburg filter showed that while iron was present in considerable quantities it was not in the sulphide form. The mud was found to consist essentially of very fine particles of coal dust mixed with the hydrates of iron and aluminum.

Although it had been understood for a long time that the drainage

from coal mines contained a considerable quantity of free sulphuric acid, an explanation of the processes by which this was formed was not generally current. Recognizing that ferrous sulphide, as pyrites, was present in the coal dust in the river, it was suspected that the "sulphur bacteria" and "iron bacteria," which work various transformations in iron compounds, might play some part in the formation of this sulphuric acid, and an experiment was made to test the hypothesis. A small quantity of coal dust or culm was pulverized in a mortar, and equal portions were placed in narrow-mouthed bottles. In each of these bottles was then placed a definite quantity of untreated river water, and, the mouths of the bottles having been closed with cotton plugs, half the number were sterilized in an autoclave at a pressure of 15 lb. for 30 min. After standing in the laboratory for about a month the bottles which had not been sterilized exhibited very different characteristics from those which had been so treated. After both sets of bottles had been shaken violently the turbidity in the set which had not been sterilized was observed to decrease very much more rapidly than that in the set which had been sterilized, somewhat after the manner that the untreated river water containing this hydrated oxide of iron acts in comparison with water which does not contain it. The water in these bottles represented, in some degree, the condition of the water in the river, except that the bottles which had been sterilized contained no living bacteria, while the others, supposedly, did, having been filled with river water which at the time of filling was known to contain a large number. The results of this preliminary experiment seemed to indicate that the change which had taken place in the unsterlized bottles was due to the action of the bacteria on the iron compounds contained in the culm.

Following out this idea, a further elaborate set of tests finally led to the conviction that a hydrate of iron was formed from the pyrites, in the presence of bacteria, while it was not formed when bacteria were absent; further, there was no evidence of the formation of soluble iron except when bacteria were present.

The reactions which took place in the bottles used in the experiments are not known, and are probably of a very complicated character, some resulting directly from, or being intimately associated with, the biological activities present, and others in all probability being spontaneous chemical reactions. It would be foreign to the subject of

this paper to enter into a discussion of these experiments in detail and of others which have since been made both on Susquehanna and on Schuylkill waters; it will be sufficient to state that at the conclusion of the Harrisburg experiments the data indicated that the bacteria occurring normally in the Susquehanna River, in the presence of organic matter on which they can live, and under proper conditions of storage, are able to transform the pyrites in the coal into compounds which furnish the material for the hydrated oxide of iron naturally occurring in the water. The indications seem to be that the sulphur in the pyrites, in contact with water, furnishes the material from which the bacteria make sulphuric acid, which, coming in contact with the iron, forms the soluble ferrous sulphate. The water containing this soluble ferrous sulphate commingles with the alkaline waters of the river farther down stream, the sulphate of iron being decomposed, the sulphur combining with the lime and magnesia to form the sulphates of these bases, and the iron being converted into the hydrate and subsequently into the hydrated oxide. This seems to be at least one method by which free sulphuric acid is formed in the drainage of the mining regions; the process also takes place to a very considerable extent in the level reaches of the river, where culm is deposited as floods recede. When the river rises, and the dirt on these flats is washed away, there is borne with it the hydrate and hydrated oxide of iron which had been formed during exposure. At the Clark's Ferry dam, some miles above Harrisburg, the water in the east channel sometimes shows this coagulated condition so plainly that frequently within an area of a few square feet one may collect a sample of either perfectly clear, or very black water.

At times of very low water there occurs along the east shore a brownish flocculent sediment composed of the hydrate of aluminum as well as the hydrate of iron. The source of the aluminum entering into the composition of this hydrate is unknown; possibly it is derived in some way from the clays.

As a result of the natural formation of the hydrated oxide of iron, and of aluminum hydrate, in the water of the river, a considerable degree of apparent self-purification takes place during the summer months. This is brought about, in the pools above the dams, by the entangling of the bacteria into flocculent masses and their consequent settlement to the bottom of the river with the other suspended

matter. The action is, on a large scale, very similar to the results of the treatment of sewage by chemical precipitation, and the active precipitants are the same as those used in the artificial process. Ultimately, a large proportion of the organisms thus carried to the bottom cease to exist in an active condition, finding their environment not suited to sustain their vitality. The apparent purification is at times so great that, although from the many cities along its course enough sewage is flowing continuously into the river to render the water at its extreme low stage offensive, yet at such times the water, as it flows past Steelton, is actually at its best, both in appearance and as to its hygienic qualities. The analyses of different samples of river water collected at Harrisburg during low water have shown as few as 10 colonies of bacteria per cu. cm., with no intestinal bacteria present; and generally, during low-water stages, when the quantity of sewage discharged into the river is greatest relatively to the stream flow, the numbers of bacteria in the river water opposite Steelton (from points far enough away to be beyond the influence of the Harrisburg sewage) are lowest and the numbers of intestinal bacteria fewest. This condition obtains, however, only during dry weather, for the first floods which come down in the fall scour out a portion of the deposits from the pools formed by the dams up stream, and then the counts show sometimes as many as 300 000 colonies of bacteria per cu. cm. of water, as well as large numbers of bacteria characteristic of intestinal discharges.

The total hardness of the river water varies with the stage of the river, yet not necessarily in proportion thereto, and is about the same, as a general thing, from bank to bank. The water is hardest at low stages when the flow is largely from springs, that on the west side being high in carbonates and low in sulphates while that on the east side is low in carbonates and high in sulphates, owing to the chemical changes taking place, as above described.

These modifying conditions destroy any relationship between turbidities, river stages, and numbers of bacteria. High bacterial counts sometimes accompany very low, as well as very high, turbidities, and, similarly, relatively low turbidities are sometimes found at very high stages of the river; further, the color of the turbidity is no indication of its total amount, as a comparison of the records of ten samples of water, each with a turbidity of 1 000 parts per million, collected at

different times, and at different parts of the river, shows all gradations of colors between bright yellow, red, dark brown, and black.

As a result of these complex conditions, the water in the river may change quickly from a moderately hard, clear water to a soft water carrying several thousand parts per million of turbidity, which, on occasions, may intermittently come down coagulated to a more or less slimy or flocculent condition with hydrated oxide of iron and hydrate of alumina. On the other hand, the river from bank to bank may be carrying its maximum load of yellow clay from the Conodoguinet and Juniata, or reddish-brown mud from the West Branch. During a general flood from the entire water-shed, all these conditions may prevail at the same time, the yellow water from the Juniata keeping on the western side, the West Branch water keeping in the central part, and the black water from the North Branch along the eastern shore, each stream occupying practically one-third the width of the river, which, opposite Steelton, is approximately four-fifths of a mile.

Although of great width, the river is comparatively shallow, the bed in that vicinity being of shale and limestone, and the depth of the water at its lowest stage being not more than 3 ft. at any point and not more than 1½ ft. for the greater part of the width. Some trouble is caused during the winter by slush-ice and anchor-ice, the formation of which is favored by these conditions.

The Steelton Filter Plant.

Those who have had experience in the filtration of polluted waters will appreciate some of the difficulties encountered in the purification of the Susquehanna River water, particularly as to bacterial purification, for the reason that the numbers of bacteria in the raw water may be increasing rapidly while the turbidity is decreasing; and hence, if the attempt is made to control the bacterial purification by operating the plant on the indications of the removal of turbidity only, the result may be anything but satisfactory.

Another complication arises from the exhaustion of the alkalinity of the raw water by the acid mine wastes, and also its reduction by dilution during heavy freshets. This, however, can readily be taken care of by adding enough lime or soda-ash to supply the deficiency in alkalinity, in case a coagulant is used in connection with the process

of purification. Among other complications is the difficulty of securing the satisfactory coagulation of the raw water when the bacterial counts are high and the turbidity is low, and also the difficulties encountered in securing a sufficiently heavy coagulum when the temperature of the water approaches the freezing point, at which times the turbidities are also usually low.

While all these difficulties can be overcome and the water can be handled successfully with a mechanical filter plant such as that installed for the City of Harrisburg, the successful operation of such a plant depends on the maintenance of a fully equipped laboratory, and on a superior grade of supervision; and, to secure these, it is essential, of course, that the revenues derived from the sale of the filtered water be sufficient to cover the additional expense.

Owing to the considerable general pollution of the river, the discharge of the sewage of Harrisburg into it only a couple of miles above Steelton, the excessive turbidity, and the enormous quantities of sand and coal delivered into the borough's distributing reservoir by the supply pumps (ultimately to enter the distribution mains, interfering with the action of meters, and clogging service pipes), the question of securing a purer and more satisfactory source of supply was brought to an issue in the winter of 1906, and the writer was commissioned to investigate and report on what could be done toward securing a new supply of a satisfactory quality, or toward purifying the existing supply. The report recommending the purification of the present supply was submitted to Councils early in 1907, and, following its adoption, the borough voted to issue $80 000 in bonds to cover the cost of the necessary works.

The plant, as built, was designed to place the securing of satisfactory results, as to purification, within the reach of the Water Department without the necessity of maintaining a laboratory, and without incurring unduly heavy construction and operation costs. The general basis of the design is as follows:

First.—The removal by subsidence of such of the suspended matters as will settle out from the raw water in about 12 min.;

Second.—The removal, by passing the water rapidly through deep, coarse-grained filters, of at least 90% of the applied turbidity, using a coagulant to secure this result when necessary, and allowing no coagulated turbidity greater than 25 parts per million to issue from

the coarse-grained filters, no matter what the turbidity of the applied water;

Third.—The filtration of the effluent of the coarse-grained filters through slow sand filters at a relatively high rate.

New Intake.

The works include a new 30-in. cast-iron intake pipe extending out about 1 500 ft. into the river to a point where observations indicated that there would be a minimum of trouble with silt, sand, and coal dust. At the point where the intake was located prior to the commencement of these improvements large quantities of sand and fine particles of coal were drawn into the intake well during floods in the river, as much as a car load a week requiring removal in order to keep the pumps in operation.

The new intake pipe was laid to proper line and grade, in cofferdams, in a trench excavated in the rock bottom of the river, the joints being poured with lead and caulked water-tight.

The intake cage consists of a reinforced concrete structure inclosing the end of the intake pipe and spreading out like a fan to give an opening in front 1 ft. high and 20 ft. long, with vertical screen bars of 1-in. round steel, spaced 6 in. apart, in two rows, and staggered. The top of the opening was located at a depth of 1 ft. below extreme low water, and the rock bottom of the river was blasted away in front of the cage, a smooth concrete apron being laid thereon to assist the current in rolling sand and coal particles along past the intake. The line of the face of the intake is parallel with the trend of the current of the river at that point.

A valve is placed in the intake pipe, at the shore end, and a connection is made with the existing 24-in. pipe leading to the suction well. The work on the intake, which was commenced on July 1st and completed early in September, 1908, was done by the Water Board by day labor, from the plans and under the general supervision of the writer.

Improvements at the Pumping Station.

New Pumps.—The plans, as approved by Councils on July 10th, 1907, called for the construction of a 3 000 000-gal. slow sand filter plant with roughing filters, to be built on a piece of ground 100 ft. wide and 265 ft. long, situated about 700 ft. east and 900 ft. north of

the pumping station. Before the construction of the works had been authorized, appropriations had been made for the installation of a new 3 000 000-gal., Heissler, compound-condensing, crank-and-fly-wheel, pumping engine at the pumping station.

In order to suit the new conditions, it was necessary to remodel the piping in the pumping station and install two 12-in. centrifugal pumps, direct-connected to simple, vertical, condensing engines of the marine type, the suction pipes of the new pumps being connected to the remodeled original suction to which the suctions of the old Deane pumps were attached. The discharge pipe from the two centrifugal pumps was piped to the filter plant, with provision for a cross-connection to the suction pipe of the new Heissler pump, so that, in case of accident to the new filtered-water well, raw water could be pumped directly into the Heissler pump and thence to the city. The Deane pumps were located in a pit sufficiently low to enable them originally to take their own suction from the intake well, but the new Heissler pump, as well as the remodeled Deane pumps now take the filtered water from a new filtered-water well constructed just outside of the pumping station.

From this brief description it will be seen that in the remodeled plant the raw water is pumped from the intake well by centrifugal pumps to the filter plant, and the filtered water is returned to the new pump well just outside of the pumping station, from which either the new Heissler or the old Deane pumps can draw the filtered water and force it through a 12-in. force main to the distribution reservoir on the hill above the town, or through the 16-in. return main directly into the distribution system, or into the reservoir and the distribution system at the same time by properly manipulating the control valves on the cross-connections. By this arrangement the construction of an additional force main to the reservoir, which would have been an immediate necessity owing to the increased consumption of water, was deferred for many years by reason of the provision of a pure water which could be pumped into the mains direct.

Raw-Water Delivery and Filtered-Water Return Mains.—The raw-water pipe leading from the pumping station to the filter plant, and the return pipe leading from the filter plant to the new pump-well are of wood-stave construction, with cast-iron specials for bends and connections to the concrete.

The staves were of Oregon pine dressed to the true circle and constructed in the usual manner with steel tongues in the end joints of abutting staves, and ½-in. round steel bands, with rolled threads and standard hexagonal nuts engaging against cast malleable-iron shoes, the bands being placed 12 in. apart, excepting at butt-joints and on the ends of cast-iron specials, where the spacing was reduced to about 4 in. The shoes and bands were dipped in a hot bath of Pioneer mineral-rubber pipe coating, and were retouched, in the trench, after cinching, with Smith's Durable Metal Coating applied with a brush.

The raw-water and filtered-water pipes have an internal diameter of 24 in., and, for the greater part of their length, are laid in one trench.

Filtered-Water Well.—The new pump well at the pumping station is of concrete. It is circular in plan, has an internal diameter of 15 ft., and a depth of 14 ft. from the floor to the underside of the reinforced roof. It has a cast-iron manhole frame and cover, the opening being 36 in. in diameter; the cover is provided with a gasket and is bolted down water-tight to the frame, which is built into the concrete.

Where the 14-in. discharge pipe from the centrifugal pumps passes through this well to form the connection with the wood-stave pipe, a by-pass connection is made by which the raw water can be discharged into the filtered-water well in case of necessity, but the valves and pipes are arranged so that any leakage in or through the valve on the raw-water pipe will drain out to the raw-water well and not contaminate the filtered water.

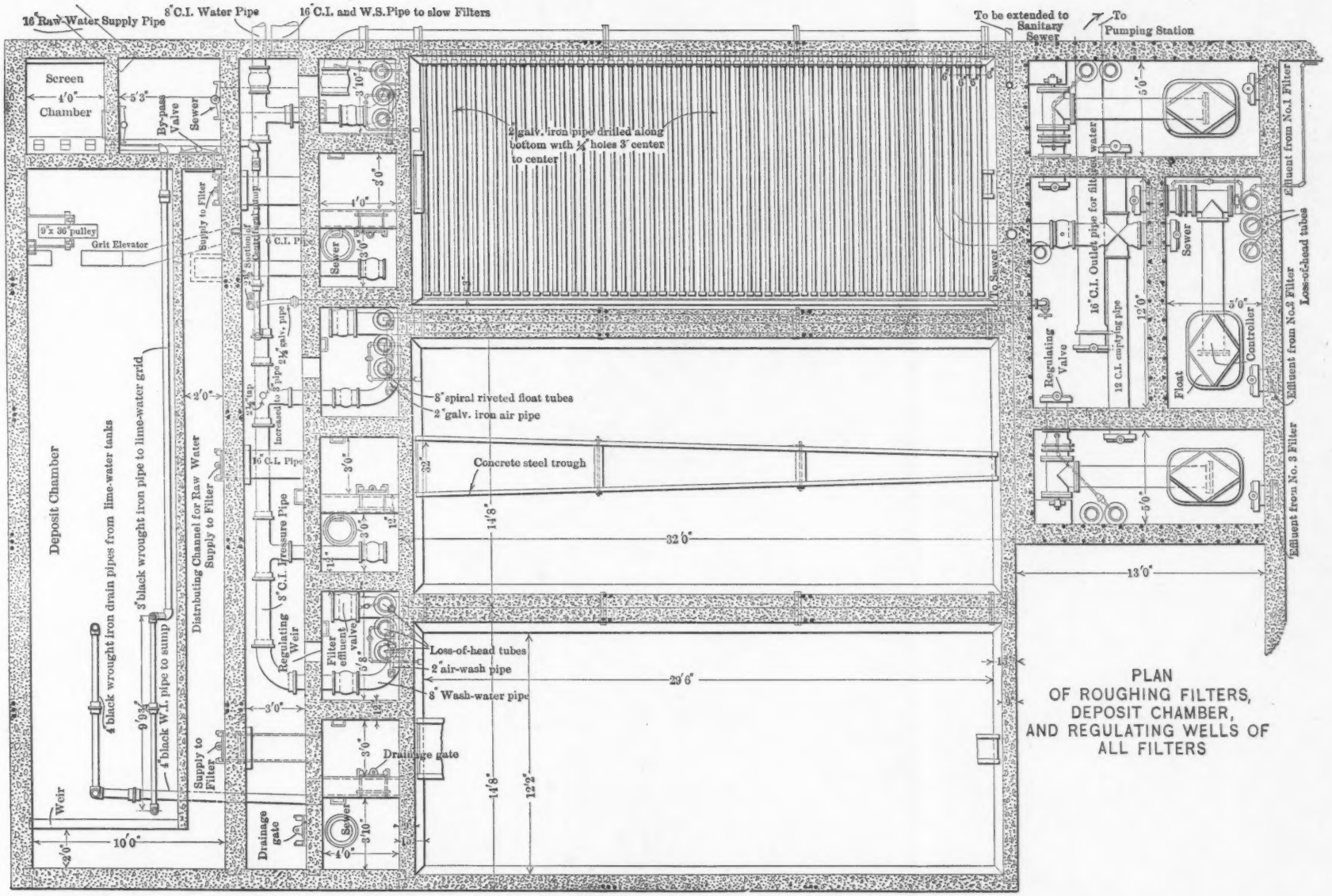
Purification Works.

Capacity of Plant.—The purification works include a deposit chamber and three roughing filters which, with their regulating wells and the regulating wells for the slow filters, occupy a space approximately 65 ft. long and 44 ft. wide; in addition there are three slow filters which occupy the remaining available space, 100 ft. wide and 200 ft. long.

The plant is designed to supply 3 000 000 gal. of filtered water per day, a proper reserve capacity being provided to permit of cleaning the filters while delivering the stated quantity of water.

Method of Purification.—The method of purification consists in

PLATE XLVII.
PAPERS, AM. SOC. C. E.
AUGUST, 1909.
FUERTES ON
PURIFICATION OF WATER.



PLAN
OF ROUGHING FILTERS,
DEPOSIT CHAMBER,
AND REGULATING WELLS OF
ALL FILTERS

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preparing the raw water for slow filtration by removing therefrom enough of the turbidity and bacteria to permit the slow filters to be operated at a relatively high rate. The object of the treatment is to prepare the water in such a way as to be able to discharge upon the slow filters, for final purification, a water of reasonably constant composition, as to turbidity and bacterial contents, throughout the entire year, keeping the maximum limits of turbidity and numbers of bacteria in the applied water down below figures which experience indicates to be desirable.

The treatment begins with the removal of the floating matters from the water by coarse screening and of the particles of sand, silt, and coal dust by subsidence. The water is then passed through roughing filters containing a deep bed of coarse-grained filtering material in which an additional portion of the remaining suspended matter (including a varying percentage of bacteria) is retained, the effluent water passing to the slow sand filters for final purification. As a general rule, when the numbers of bacteria in the raw water are relatively low and when the turbidity does not exceed 50 parts per million, the roughing filters produce an effluent of satisfactory quality without the use of artificial coagulation; but when the turbidity of the raw water is higher than 50 parts per million, or when it is caused by particularly fine particles, or when the bacteria number more than about 5 000 per cu. cm. in the raw water, a sufficient quantity of coagulant is mixed with the incoming raw water, as it enters the deposit chamber, to produce a rough filtered effluent with a turbidity of not more than 10% of that of the applied water, and not more than 25 parts per million, in any case. Adherence to these limits, which are as yet only tentative, will apparently give sufficiently long runs of the roughing filters to permit of easily keeping the plant in operation, and sufficiently long periods of operation of the slow filters to enable them to pass a satisfactory quantity of filtered water between scrapings.

The period of subsidence allowed in the deposit chamber, for the maximum capacity for which the plant is designed, is 12 min. The roughing filters are designed to operate at a rate of 172 000 000 gal. per acre of filter surface per day when delivering 1 500 000 gal. per filter, three filters being provided so that two can yield the full 3 000 000 gal. while the third is being washed. The net area of the sand surface of each slow filter is 0.1446 acre, and when delivering 1 500 000 gal.

per day the filter will operate at a rate of 10 373 000 gal. per acre per day, two filters yielding 3 000 000 gal. daily with the third in reserve for cleaning.

Deposit Chamber.—The raw water is received at the filter plant at the bottom of a rectangular compartment in one end of the reinforced concrete deposit chamber, Plate XLVII, a valve-controlled connection being provided between the bottom of this compartment and the sewer. One outlet from the upper part of this compartment leads to the deposit chamber and a second leads directly to the channel feeding the water to the roughing filters. By using one or the other of these outlets the water may be passed through the deposit chamber, or, if desired, the deposit chamber may be by-passed and the raw water delivered directly to the roughing filters. After leaving the inlet chamber, on its way to the deposit chamber, the water first passes vertically downward through a coarse screen composed of parallel lines of 2-in. planks standing on edge, horizontally, and spaced about 8 in. apart from center to center transversely to the direction of the entering water; between these planks, and in a vertical plane, one above the other, are the horizontal perforated pipes through which the lime-water and coagulant solutions are admitted to the raw water.

The raw water is admitted to the deposit chamber in the manner described, in order to use up the entering velocity head and permit the water to pass through the deposit chamber quietly and thus drop out as much as possible of the suspended matter in the short time allowed therefor.

Grit Elevator.—Near the end at which the raw water enters, a grit elevator is installed to remove continuously the sand and coal particles which may collect in the deposit chamber. The elevator, Plates XLVII and XLVIII, which has an estimated capacity of 20 tons per 24 hours, consists of an 8-in., 6-ply rubber belt carrying perforated, galvanized-iron buckets, 6 in. long, 4 in. wide and 4 in. deep, at intervals of 2 ft. The belt travels at the rate of 25 ft. per min., and the buckets discharge into a hopper arranged to fit into the end of a 6-in. cast-iron pipe leading to one of the sewer compartments of a roughing filter.

The deposit chamber is 10 ft. wide and 42 ft. long, including the entrance chamber and screen box, and at full capacity the water flows through it with a depth of about 9.2 ft. and a velocity of 0.05 ft.

PLATE XLVIII.
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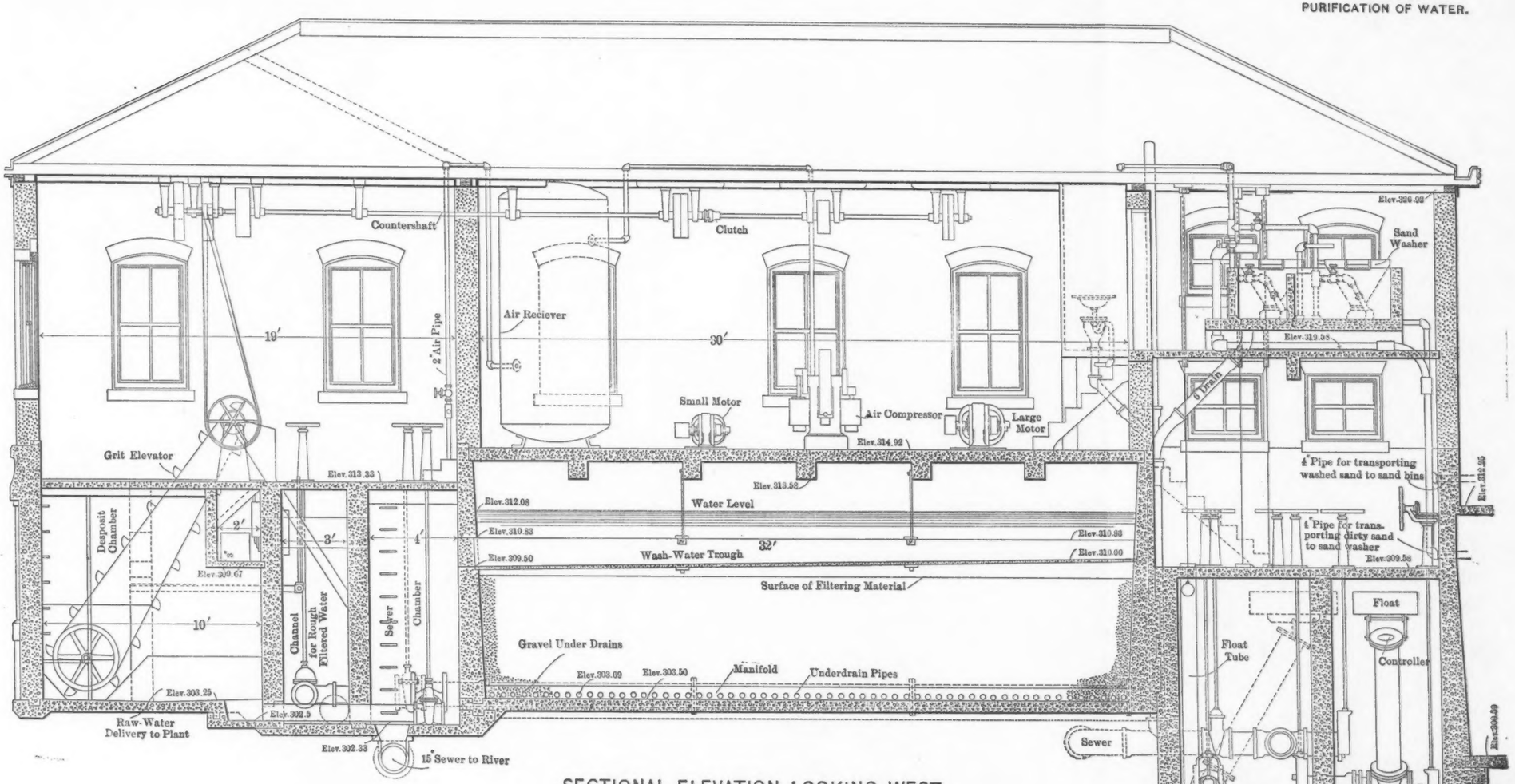
FUERTES ON
PURIFICATION OF WATER.



SECTIONAL ELEVATION LOOKING WEST

UNIV. OF MICH.

per sec., passing over a weir at the farther end into a channel leading to the inlet openings to the roughing filters.

Chemical-Mixing Tanks.—Vertically above the deposit chamber, and carried on heavy reinforced concrete beams spanning the chamber, stand the mixing tanks, Plates XLIX and L, for the lime-water and coagulant solutions. These tanks are of monolithic reinforced concrete. In each tank there is a wooden dissolving rack with an open bottom carrying wooden frames provided with copper screens having 20 meshes per inch, reinforced by a coarse copper screen having 4 meshes per inch, with No. 11 bars. Each tank is provided with a system of $\frac{5}{8}$-in. perforated copper pipes at the bottom connecting with a line of galvanized-iron pipe leading from above the tops of the tanks to the air receiver, to provide for the agitation of the solution after it has been mixed. A galvanized-iron steam connection, with a copper branch reaching to the bottom of the coagulant tanks, is provided for warming the solution during cold weather. The solution is mixed in the usual manner. With a 2% solution, two tanksful will furnish a dose of 2 gr. per gal. for a consumption of 3 000 000 gal. daily. The coagulant solution is drawn from the mixing tanks through 2-in. copper pipes arranged with swivel-joints to permit of decanting the solution at any desired depth.

Coagulant-Measuring Box.—Through the copper outlet pipes the solution runs to the coagulant-measuring box, Fig. 1, and Fig. 1, Plate LI. This is a reinforced concrete box with a plate-glass front in which are drilled three $\frac{1}{4}$-in. and three $\frac{1}{2}$-in. orifices, all at the same elevation and spaced 4 diameters apart and 4 diameters of the $\frac{1}{2}$-in. holes from the bottom and side edges of the plate. This glass plate is 11 in. wide, $\frac{1}{4}$ in. thick, and 28 in. high, and the holes are countersunk one-half the depth of the plate from the outside. On the back of the plate is etched a graduated scale reading to hundredths of a foot for a height of 2 ft. above the centers of the orifices. The plate is fastened to the front of the orifice box on soft rubber gaskets, made of tubing, by brass plates and expansion bolts secured in the concrete. The depth of the solution in the orifice box is regulated with a bronze lever valve and float arranged to admit the solution near the bottom and permit the regulation of the depth in the box by changing the height of the float. The float is of copper, 10 in. in diameter, with a flat bottom and a coned top, sliding on a $\frac{1}{4}$-in. brass rod, the rod being

hinged to the lever of the valve, and the height of the float being adjusted on the rod with a bronze set-screw having a milled head.

Each orifice is provided with a soft rubber cork, or stopper, attached to a handle consisting of a piece of brass pipe with a cap on one end and an elbow on the other, the cork being fastened to a nipple entering the elbow; there is a rack, with thumb-screws, across the top of the orifice box, for holding the handles where desired.

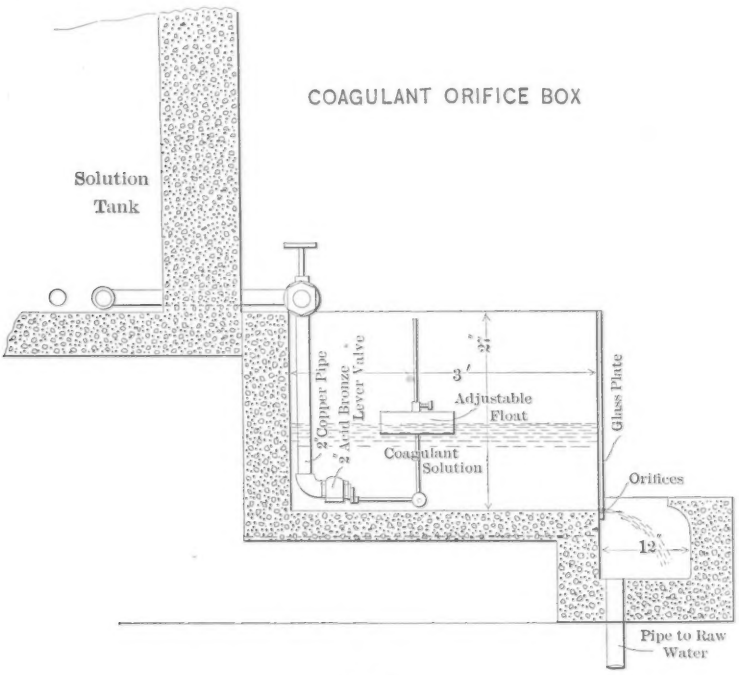
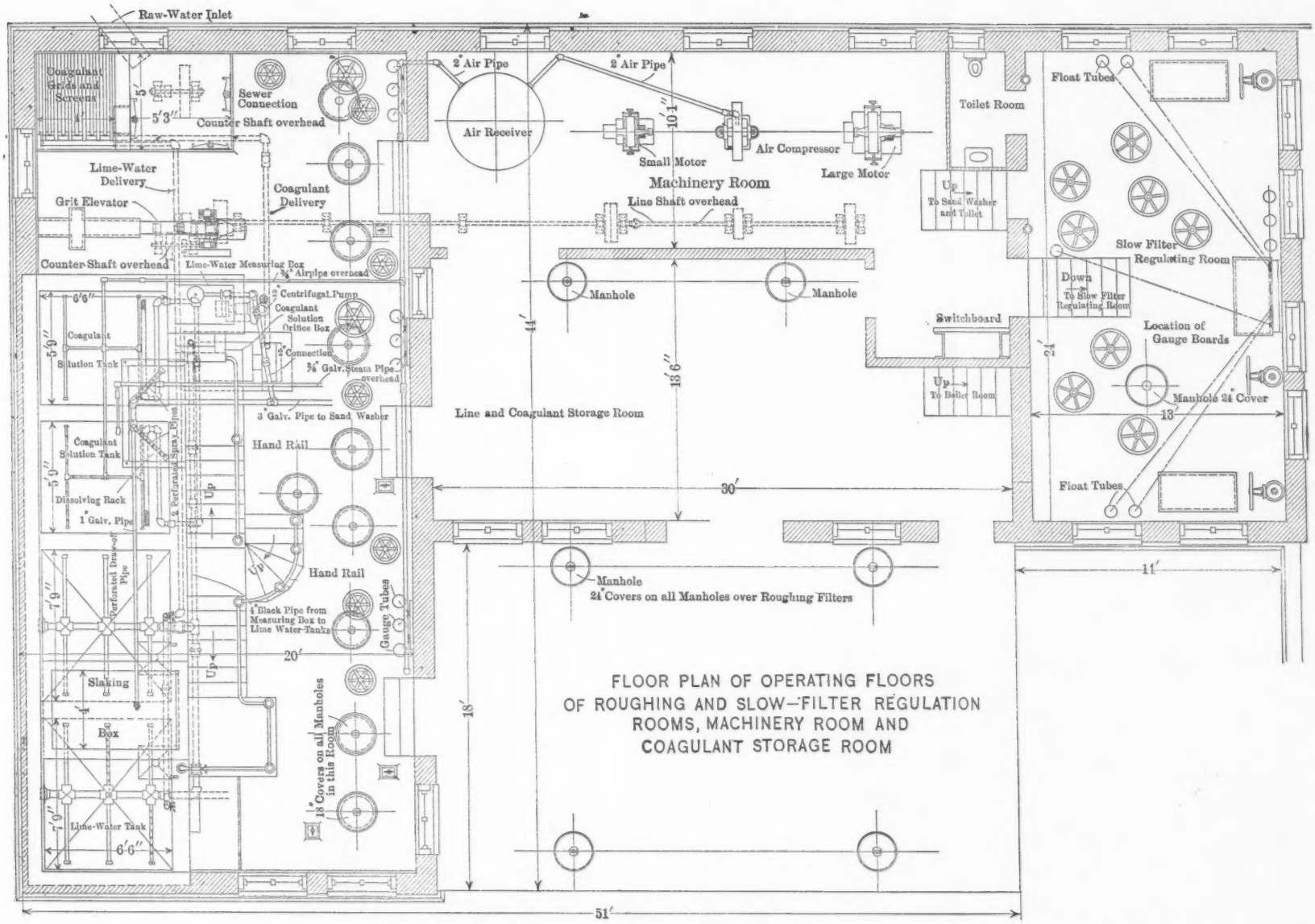


FIG. 1.

The coagulant solution flows through the orifices in the glass plate to a small concrete box, from which a 2-in. copper pipe connects with the perforated pipes between the screen bars at the entrance to the deposit chamber.

Lime-Water Mixing Tanks.—The lime-water apparatus consists of two tanks for making the lime-water (Plates XLIX and L), a slacking box, and an orifice box for measuring the quantity of water to be supplied to the lime-water tanks. Lime being only slightly soluble in water, a relatively large quantity of solution is required



FLOOR PLAN OF OPERATING FLOORS OF ROUGHING AND SLOW-FILTER REGULATION ROOMS, MACHINERY ROOM AND COAGULANT STORAGE ROOM

unless the lime be added as cream of lime. In the Steelton plant the apparatus is designed to make use of a saturated solution of lime-water.

The lime is slaked in a reinforced concrete box supported on top of the partition separating the two lime-mixing tanks, a sluice-gate being provided on each side for emptying the milk of lime through funnels into pipes leading down to the bottom of the lime-mixing tanks. The latter are rectangular in plan, the bottom of each being in the form of an inverted frustum of a pyramid. From the bottom of each a pipe leads to the sewer, with a connection immediately below the bottom of each tank leading to the lime-water measuring box which stands on brackets above the coagulant-mixing tanks and receives its water through a float and lever valve (with spindle and float as described for the coagulant orifice box) either from the city mains or from a 2-in. centrifugal pump taking its suction from the channel containing the rough-filtered water. Valves in the various pipes permit of cleaning the tanks and pipes, and the admission of water to either of the tanks as desired. The lime-water orifice box is also of reinforced concrete, with a plate-glass front, $\frac{1}{2}$ in. thick, 20 in. wide and 28 in. long, containing two $\frac{1}{2}$-in., two 1-in., and one $1\frac{1}{2}$-in., orifices; there is also a graduated scale etched on the back of the glass, the details of the arrangements being similar to those for the coagulant orifice box. The effluent from the solution-measuring box issues vertically into the mixing tanks through the center of the bottom, rising as a saturated solution through the milk of lime, which has been mixed and deposited in the bottom of the tank. The quantity of water rising through the tank is varied from time to time in accordance with the different conditions of the raw water and the varying consumption of water by the borough, the maximum rate, for a dose of 2 gr. per gal., for a consumption of 3 000 000 gal. daily with both tanks in service, being such that the water would rise through the tanks at the rate of 7 in. per hour if the outlet valves were closed. The water, having picked up its lime at the bottom of the tank, is skimmed off just below the surface by a system of perforated, horizontal, galvanized-iron pipes.

Application of Chemicals.—For mixing the lime-water with the raw water, six lines of $1\frac{1}{2}$-in. galvanized-iron pipes, perforated with double rows of $\frac{5}{16}$-in. holes $3\frac{1}{2}$ in. apart, screwed into a 4-in. galvanized-iron manifold connected with a loose-sleeve joint to the lime-water

supply pipe, lie between the wooden bars of the screen at the entrance to the deposit chamber. The copper pipes for distributing the coagulant solution consist similarly of six lines of 1-in. pipes perforated with double rows of $\frac{3}{16}$-in. holes spaced $3\frac{3}{8}$ in. apart and fastened to a $2\frac{1}{2}$-in. copper manifold; they lie vertically below the lime-water mixing pipes.

Roughing Filters.

The raw water, after passing through the deposit chamber, or in case it is by-passed around the deposit chamber, enters a channel, running lengthwise along one side of the deposit chamber from the

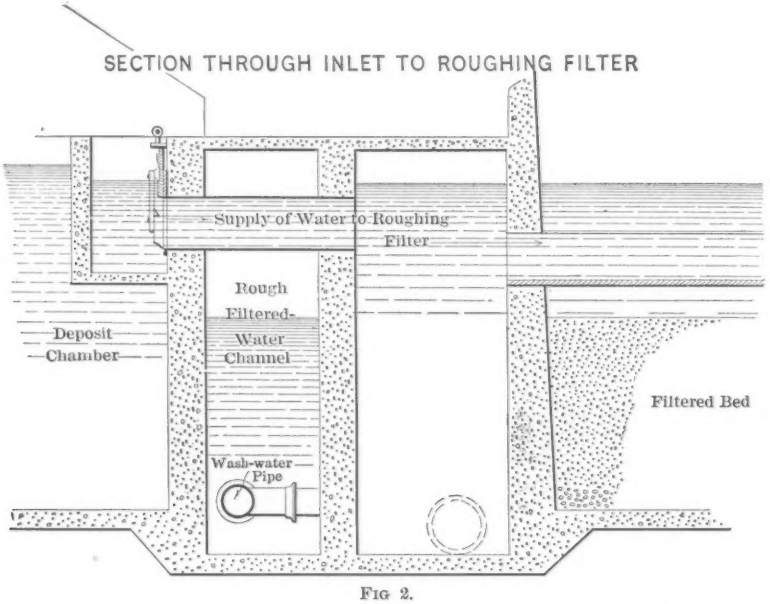
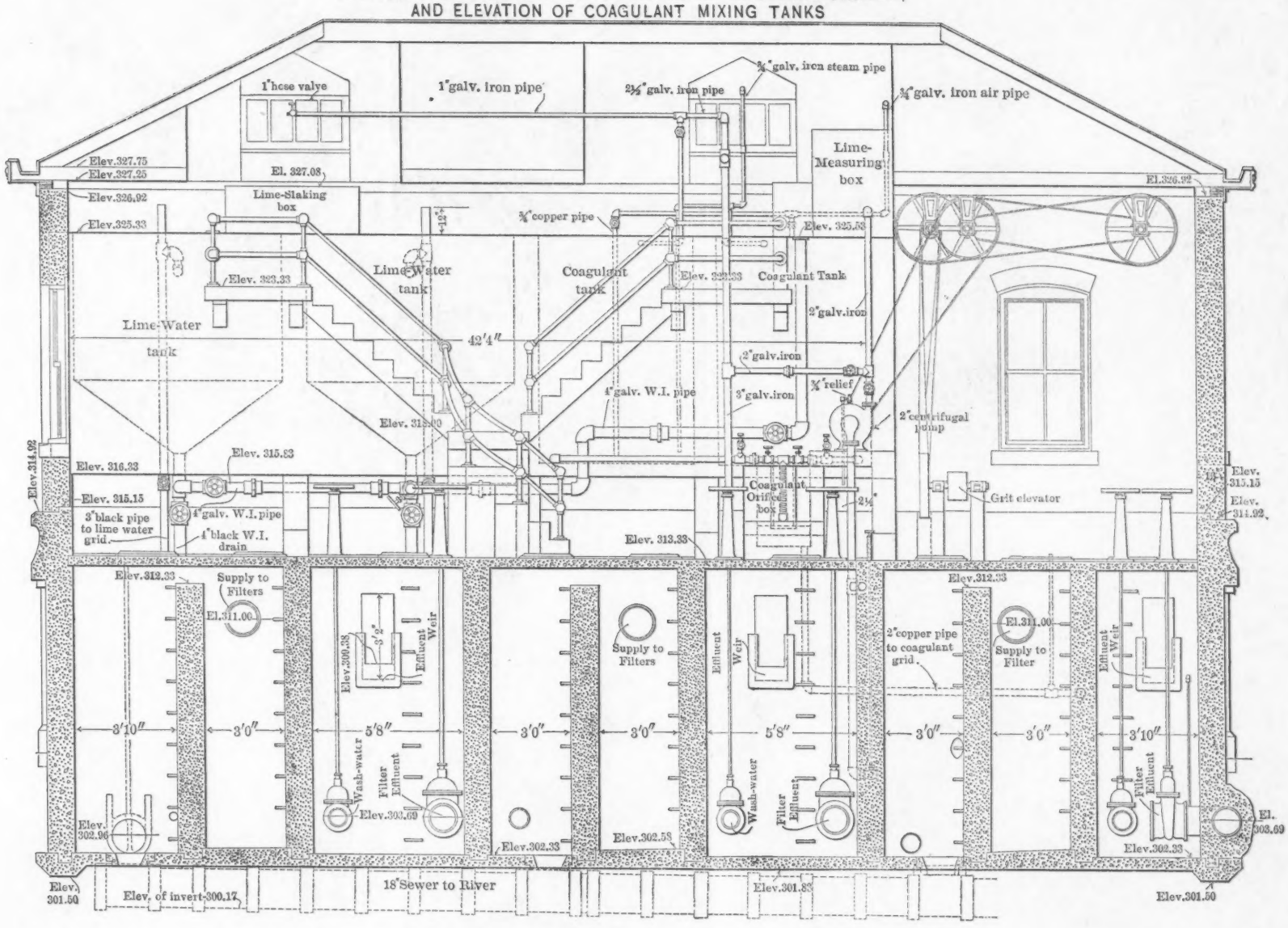


Fig 2.

weir at the east end to the entrance chamber at the west end, the side and bottom of this channel being supported by vertical steel bars built in the concrete and passing continuously through the side of the channel and the beams spanning the deposit chamber and carrying the mixing tanks. From this channel, Plate XLVII, the water is taken to each of the three roughing filters through 16-in. cast-iron pipes controlled by 16-in. sluice-gates operated with chain-hoists. Between the ends of the three filters and the east side of the deposit chamber are arranged a series of chambers, three for each filter, containing connections to the sewer, to the filtered-water effluent pipe, and to the raw-water supply, a connection being made at the bottom, between this

SECTION THROUGH REGULATING WELLS OF ROUGHING FILTERS, AND ELEVATION OF COAGULANT MIXING TANKS



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latter chamber and the chamber leading to the sewer, the purpose of which is to permit the dirty water resulting from the washing of the filters to flow backward through this inlet chamber to the sewer leading to the river. Between the row of regulating chambers for the filters and the deposit chamber is a narrow channel, 3 ft. wide, and about 10.5 ft. deep, into which the rough-filtered water is discharged from the roughing filters, and from which the supply pipe leads to the slow filters.

The 16-in. supply pipes to the roughing filters cross the channel containing the rough-filtered water, and enter the supply chambers immediately in front of the center of each roughing filter. The wash-water troughs, which serve the purpose of distributing the incoming water over the filters during operation, as well as of removing the wash-water when the filters are being cleaned, are suspended from the roof beams of the roughing filters by 1-in. bolts engaging saddle pieces of flat steel, over the top and under the bottom of the troughs, with jamb-nuts at top and bottom to permit of adjustment.

The roughing filters are each 12 ft. 2 in. wide and 29 ft. 6 in. long at the bottom, the sides and ends battering outward 6 in. in the height of the filter, the outside surfaces of the walls being vertical. The side walls and roofs of the filters are of reinforced concrete, and the bottoms of concrete without reinforcement.

Roughing-Filter Underdrains.—In each filter there is an underdrainage system, similar in principle to that designed by the writer in 1902 for the Harrisburg Filter Plant. It consists of 59 parallel 2-in. galvanized-iron pipes, extending across the width of the filter and drilled along the bottom with $\frac{1}{4}$-in. holes 3 in from center to center, the pipes entering a manifold made of 12-in. flanged pipe built into the concrete wall on one side of the filter and terminating in the effluent regulating chamber in front of the filter in a 12-in. gate-valve operated by hand from the floor above. The 2-in. galvanized-iron pipes are placed in the filter so that their bottoms stand 1 in. above the floor of the filter, and are surrounded with a 4-in. layer of gravel composed of stones which will pass through a screen having $\frac{3}{4}$-in. meshes in the clear between the wires and remain on a screen having $\frac{1}{4}$-in meshes in the clear between the wires, and containing no particles finer than $\frac{1}{4}$ in. in largest dimensions. Upon this rests a second layer of fine gravel, 3 in. thick, all the particles of which will pass through a screen having $\frac{1}{4}$-in. meshes, but will remain on a standard brass sieve having

12 meshes per lin. in., and containing not more than 2% of particles which will pass through a standard sieve having 15 meshes per lin. in. These specifications were written for local materials, and might require modification for other materials of different subsiding values.

Filtering Materials in Roughing Filters.—The filtering material in the roughing filters consists of a layer of fine anthracite coal screenings, 5 ft. thick, prepared from particles of fine coal washed down the river from the culm piles of the mines on the North Branch water-shed during floods and recovered from the river in the neighborhood of Harrisburg and Steelton by centrifugal pumps. Considerable care was required in the selection of the raw material, as it was desired that the finished product should have an effective size of about 1 mm. and a uniformity coefficient as low as practicable without making the cost of preparation too great. The material as received at the plant contained considerable moisture in the pile and was prepared by casting over a screen having 4 meshes per in., the actual separation with the wet coal being practically the same as would be obtained by screening the dry material through a sieve having 6 meshes per in. Each of the three filters contains 5 ft. in depth of this screened coal, which, as prepared, has an effective size of 1 mm., a uniformity coefficient of about 2.4, and does not contain more than 2% of particles which would pass through a screen having 30 meshes per in. In preparing the river coal, 144 tons of the finished product, weighing about 53 lb. per cu. ft., was required, to secure which it was necessary to screen 176 tons, rejecting 32 tons of coarse particles.

Roughing-Filter Controller.—The rate of filtration of the roughing filters is controlled by regulating the depth at which the rough-filtered water flows over the edge of standard bronze weir-plates, 12 in. long, Plate L and Fig. 3, the weirs being placed in orifices in the front walls of the effluent regulating chambers of the filters, the rough-filtered water falling over the weirs and into its channel, from which it is conducted to the slow filters, the elevation of the weirs being fixed so as to allow losses of head up to about 2½ ft.

Loss-of-Head and Rate Gauges.—The rate of filtration and loss of head are indicated, for each filter, by gauges and indexes operated by spherical, seamless, spun-copper floats, resting on the water in three 8-in. spiral riveted tubes, Plate XLVII and Fig. 3, placed in the regulating chamber of each roughing filter.

FIG. 1.—ROUGHING-FILTER OPERATING ROOM.

FIG. 2.—FORMS FOR ROOFS OF SLOW FILTER.

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The rate gauge was graduated at the plant from the actual measured discharge over each weir in stated periods of time, and at different depths of flow, as indicated by the float gauge. The loss-of-head gauge was divided into feet and hundredths.

The gauge boards were made of strips of poplar, painted with three coats of white-zinc paint, graduated by hand with indelible India ink, the figures and letters being lead pattern-makers' letters attached to the gauge boards with shellac, on the first coat of paint, and painted white with the board before the latter was graduated. After graduation the tops of the letters were blackened, and the board was given two coats of spar varnish and stiffened by small brass cleats

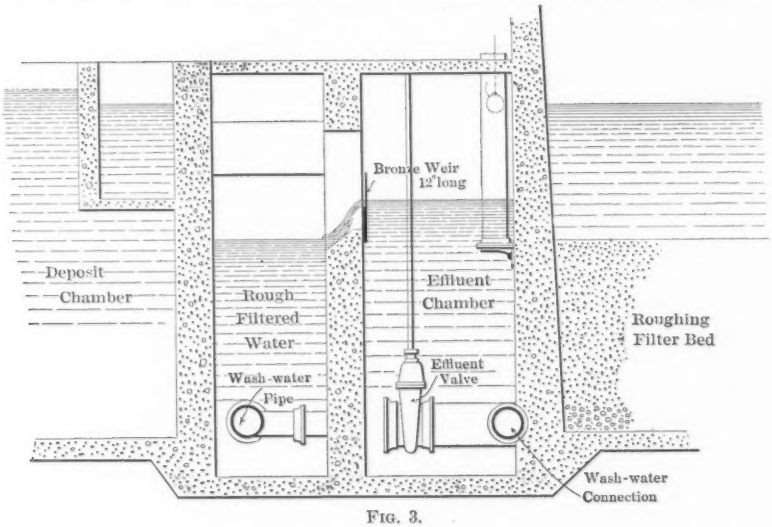


FIG. 3.

screwed to the backs at the top and bottom, the cleats being provided with ears on each side drilled with holes, $\frac{1}{2}\frac{1}{4}$ in. in diameter, parallel to the back of the board and lengthwise of the same. The gauges were strung on parallel guides formed of No. 20, B. & S. gauge, phosphor-bronze wire fastened to screws at the top and bottom in the head and foot blocks.

Roughing-Filter Washing Devices.—Provision for washing each roughing filter was made through an 8-in. connection with a 12-in. pipe laid especially to bring filtered water to the filter plant from the force main leading from the city pumping station to the service reservoir, with a cross-connection also to the 16-in. return main, so that

water may be drawn from either one or the other of these two pipes as desired. The 12-in. pipe divides, on reaching the filter plant, an 8-in. branch, Plate XLVII, entering the roughing-filter plant at the bottom of the channel which receives the rough-filtered water, with connections with the underdrain of each filter back of the effluent control valve, each of these connections being controlled by its own valve operated by a hand-wheel from the operating floor above. In addition, one other 8-in. branch is taken off from the 8-in. wash-water main and terminates in the sewer chamber of the central filter for use in flushing the sewer, if necessary.

In addition to the wash-water, provision is made for scouring the beds of the roughing filters with air admitted to the underdrains through 2-in. galvanized-iron pipes leading from the air receiver and provided with control valves and pressures gauges. The air pipes connect with the wash-water pipes in the bottom of the filtered-water effluent chamber of each filter between the wash-water control valve and the effluent underdrain.

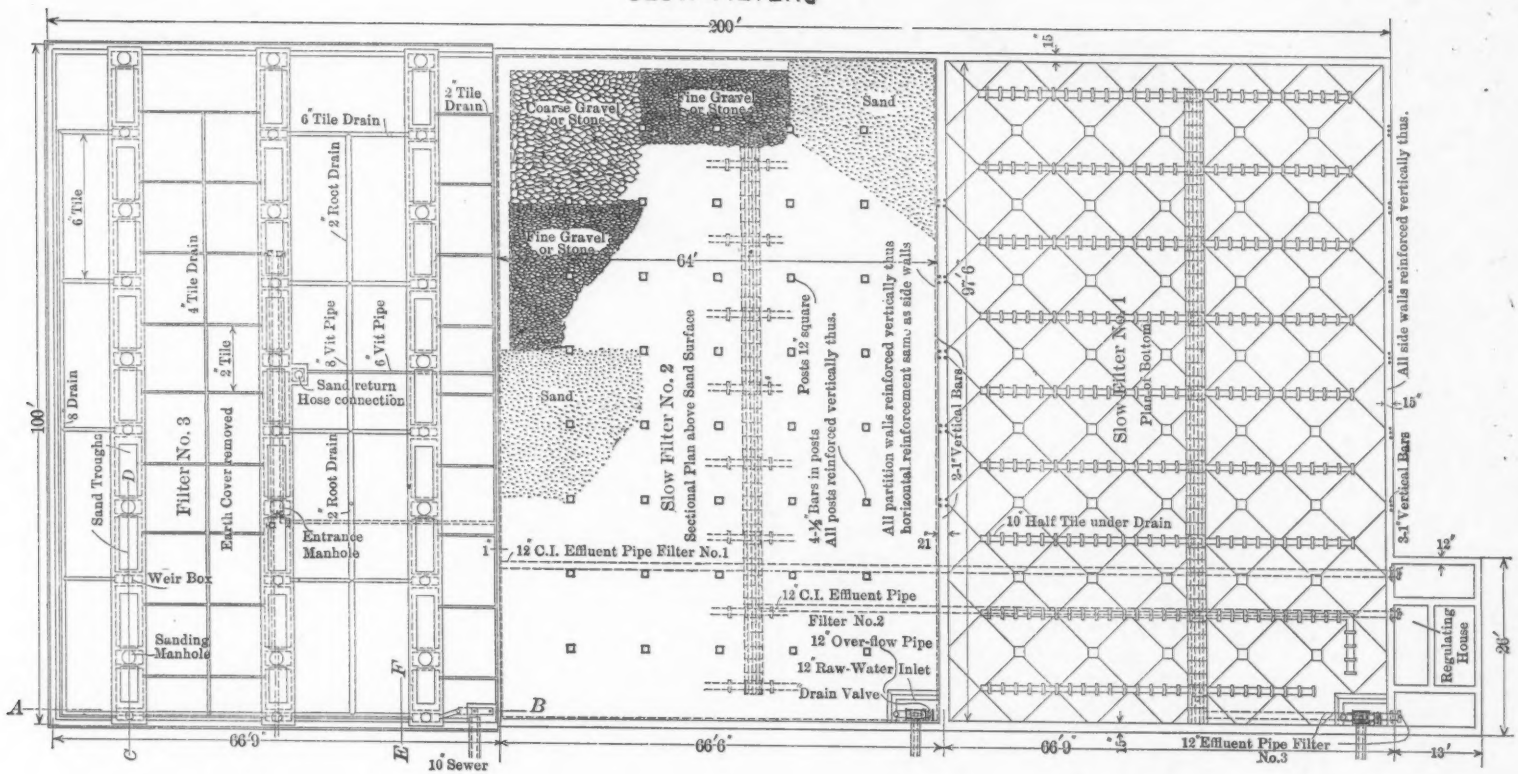
Sewer Connection.—Under the regulating chambers of the roughing filters there is an 18-in. vitrified pipe sewer, Plates XLVII and L, laid in concrete and extending to the river, a distance of approximately 1 000 ft. Into this sewer all parts of the plant can ultimately be drained.

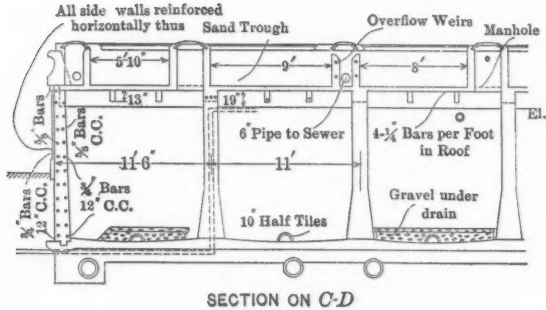
The floor of the operating room of the roughing filters forms the roof over the various regulating chambers and the channel for the rough-filtered water; and the manholes providing for entrance into the various chambers are covered with light, 20-in., circular, cast-iron covers with ring-frames built into the concrete, manhole steps being provided in the walls.

General Water-Supply Pipes.—The water supply for mixing the coagulant solution, for supplying the steam-heating plant, the toilets, and the sand-washer plant of the slow filters, is taken from the 8-in. wash-water main in the effluent channel of the roughing filters by a vertical 3-in. galvanized-iron pipe rising above the ceiling of the operating room and extending lengthwise of the roughing filters to the sand-washer room in the second story of the building over the regulating chambers of the slow filters. The branches for the different fixtures are taken from this main line where required.

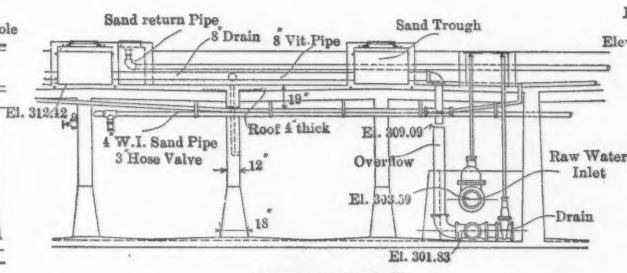
Operating Platform for Chemical Tanks.—From the floor of the operating room of the roughing filters, reinforced concrete stairways,

SLOW FILTERS

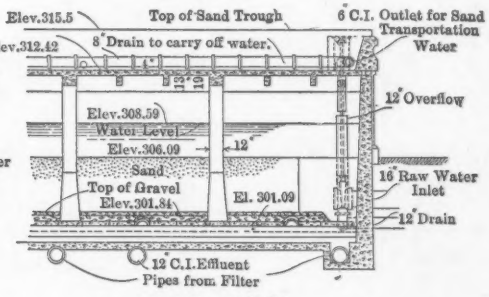




SECTION ON *C-D*



SECTION ON *A-B*



SECTION ON *E-F*

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Plate L, lead up on either hand to the elevated platform in front of the coagulant-mixing tanks, and the lime-water mixing tanks. These are provided with iron-pipe railings, painted black and varnished.

Centrifugal Pump.—The centrifugal pump, Fig. 1, Plate LI, which supplies the water through the measuring box to the lime-mixing tanks, has a 2-in. discharge pipe, and a 2-in. suction pipe, extending down into the channel containing the rough-filtered water, having on its lower end an elbow, short nipple, and gate-valve with extension stem reaching up above the floor of the operating room and there provided with a hand-wheel. Just below the floor level of the operating room a 2-in. horizontal branch extends through one of the roughing filters to the slow-filter operating room where a hose-valve for 2-in. suction hose is provided in order to permit of using the centrifugal pump to empty the regulating wells of the slow filters below the elevation to which they could be drained naturally.

Electric Motors.—Power to operate the machinery in the plant is derived from the electric current supplied by the York Haven Power Company, and is of the alternating-current type delivered at a voltage of 220 at the switch-board. The motors, Plate XLIX, of which there are two, manufactured by the Westinghouse Electric and Manufacturing Company, one of 5 h.p. and the other of 15 h.p, are of the alternating-current type, wound for 3-phase, 60-cycle, 200-volt terminal voltage, capable of standing an overload of 25% for 2 hours without injury, and provided with oil-immersion starting boxes. A marbleized-slate switch-board, mounted with volt meter, ampere meter, watt meters, circuit breakers, ground detectors, pilot lights, clock, and main and circuit switches for the different services required, stands in the machinery room.

Lighting.—All the filters and different parts of the plant are provided with electric light fixtures wherever light may be required, and all the rooms of the building containing the machinery and the regulating chambers of the different filters are also provided with gas jets.

Air Compressor.—The air compressor for supplying compressed air for scouring the filtering materials in the roughing filters, and for agitating the coagulant solution, stands between the two motors, and can be operated from either one. It is an Ingersoll-Rand compressor, known as the Imperial Type 11, and consists of two cylinders standing vertically, with single-acting plungers driven by crank shafts on opposite sides of a heavy fly-wheel which also serves as the belt pulley.

There is provided on the outlet from the compressor an unloading device, so-called, which, when the pressure reaches the maximum for which the machine is intended, allows the excess pressure to be relieved automatically.

The air compressor has a capacity of 39 cu. ft. of free air per min. at a speed of 200 rev. per min., and is capable of delivering the air at that rate under a pressure of 100 lb. per sq. in. The air cylinders are water-jacketed, with hooded ends, the cooling water being discharged into the trough of the filter below; the air valves are of the poppet type, and work vertically.

Air Receiver.—A 2-in. galvanized-iron discharge pipe, with a brass check-valve near the compressor, leads from the compressor to the air receiver, which is a vertical cylindrical tank, with domed ends, 5 ft. in diameter, and 12 ft. high. The shell is of flange-steel, is furnished with a manhole and a cast-iron base, and is proportioned to stand a working air pressure of 110 lb. per sq. in. and remain air-tight under that pressure. The receiver is provided with connections for the 2-in. galvanized-iron discharge pipe from the compressor and the 2-in. galvanized-iron air pipe leading from the receiver to the air wash-pipes of the roughing filters, and has also a 1¼-in drip at the bottom, provided with a double valve.

The receiver is provided with a safety valve set at 105 lb. per sq. in., a gauge to indicate the pressure in the receiver, and a 1¼-in. Foster reducing valve capable of discharging the entire contents of the receiver in 2 min. at a uniform pressure on the filter side of the valve of about 5 lb. per sq. in., the pressure of the receiver falling, at the same time, from 100 to about 10 lb. per sq. in.

The two motors, the air compressor, and the air receiver stand on foundations built monolithic with the roof of the west roughing filter, and no vibration is apparent when these machines are operating.

Countershaft.—The countershaft to which the motors and air compressor are belted is provided with a jaw coupler between the belt pulleys from the two motors, and is extended through the south wall of the machinery room into the regulating room of the roughing filters, Plates XLVIII and L, where additional countershafts are installed in order to reduce the speed as required for the operation of the grit elevator, and to increase the speed for the operation of the centrifugal pump supplying the water for the lime-water solution.

FIG. 1.—TOPS OF SLOW FILTERS DURING PLACING OF EARTH AND TOP-SOIL COVERING.

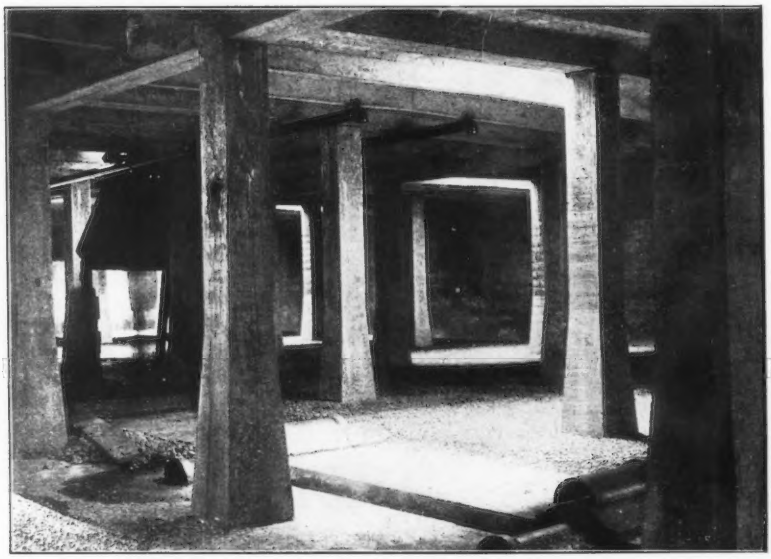

FIG. 2.—INTERIOR OF SLOW FILTER, SHOWING PLACING OF UNDERDRAINS AND GRAVEL LAYERS.

Superstructure.—The building covering the operating room of the roughing filters, the coagulant storage room, the machinery room, and the regulating room of the slow filters, Fig. 2, Plate LIV, is L-shaped, one leg of which is 44 and the other 65 ft. long, measuring on the long sides, the two wings being, respectively, 21 and 26 ft. wide. The outer faces of the side walls are of red brick, white brick being used on the inside faces. The building is surmounted by a slate roof on a timber framing, with copper ridge, finials, gutters, and down-spouts, the down-spouts discharging into the troughs of the roughing filters, or into the deposit chamber, as necessary.

Sampling Devices.—In front of each roughing filter, and by the side of the entrance chamber for raw water, a small concrete bracket, Fig. 1, Plate LI, and Fig. 1, Plate LV, containing a copper funnel in the center, is fastened to the wall, and carries a nickel-plated goose-neck on the discharge pipe of a small rotary hand-pump, the mechanism of which is submerged in the water immediately below the shelf. The pumps are operated by extension shafts carrying nickel-plated hand-wheels. These pumps are used for obtaining samples of the raw water as it enters the plant and of the effluent from each roughing filter.

Slow Filters.

The rough-filtered water is conducted to the slow sand filters through a line of 16-in. wood-stave pipe with 12-in. connections to each slow filter, controlled by sluice-gates with stems extending to the tops of the filters under a manhole.

Area.—The slow filters, Plate LII, and Fig. 2, Plate LIV, of which there are three, with a total net area of filtering surface of 0.4340 acre, lie side by side immediately north of the roughing filters. They are each 97 ft. 6 in. long, the two end filters being 64 ft. 7½ in., and the central filter, 64 ft. 9 in. wide, all dimensions being measured inside at the bottom, and all interior wall surfaces battering out toward the top 6 in. in their height. The thickness of the outside walls of the filters is 15 in. at the bottom, and 9 in. at the top; the thickness of the division walls is 21 in. at the bottom and 9 in. at the top. The walls are reinforced horizontally and vertically to take the necessary strains, and are built in sections ending against headers, with copper strips in all vertical joints to cut off any possible leakage at such points.

The floors of the filters are laid in blocks, and form inverted groined arches, with the columns supporting the roof standing on the high points.

Roof, Side Walls, and Floors.—The roof of each filter is supported on forty reinforced concrete posts, 12 in. square from the roof to a point 4¼ ft. from the bottom, battering out to a width of 18 in. on each side at the floor level. Longitudinally of each filter, reinforced concrete beams, 19 in. deep, run across the tops of the columns in alternate rows, while transversely of the filters, beams, 12 in. deep, run across the tops of all the columns in each row; on the intermediate lengthwise rows of columns the functions of beams for carrying the roof are performed by the deep sides of the troughs intended for the storage of sand removed from the filters during the periodical scrapings, the roof of the filter and the intersecting roof beams being tied to, and suspended from, the side walls of these troughs by vertical steel rods.

The reinforcement of the side walls of the filters consists of three vertical 1-in. bars, opposite each panel point of about 11 ft., running from top to bottom of the wall, the horizontal reinforcement consisting of ¾-in. bars, 12 in. from center to center, for the lower 8 ft. in depth, and ⅝-in. bars, 12 in. from center to center, for the remainder of the height, bars being used in both faces of each wall. The side walls rest in grooves formed along the outer edge of the floor slabs when the latter were laid. In laying the floor slabs, no particular precaution was taken to prevent leakage, further than to form grooves in each side of each slab as it was laid, allowing plenty of time for the concrete to harden and contract, before laying the next slab against it, and using a grillage of ½-in. bars, 4 ft. long, in the concrete of the floor under the bottom of each post supporting the roof. The grooves in the edges of the floor blocks were made V-shaped so that the settlement of any particular block would tend to wedge the joints tight.

Sand Troughs.—The sand troughs, Fig. 2, Plate LI, and Fig. 1, Plate LIII, on top of the filters, of which there are three to each filter, are divided by cross partitions into boxes from 8 to 9 ft. long, separated at one end by weirs and at the other by manholes opening into the filter. The boxes are arranged so that the weirs of two adjacent boxes stand 12 in. apart, forming a channel crosswise of the sand box, from one end of which leads a 6-in. drain-tile connecting with a

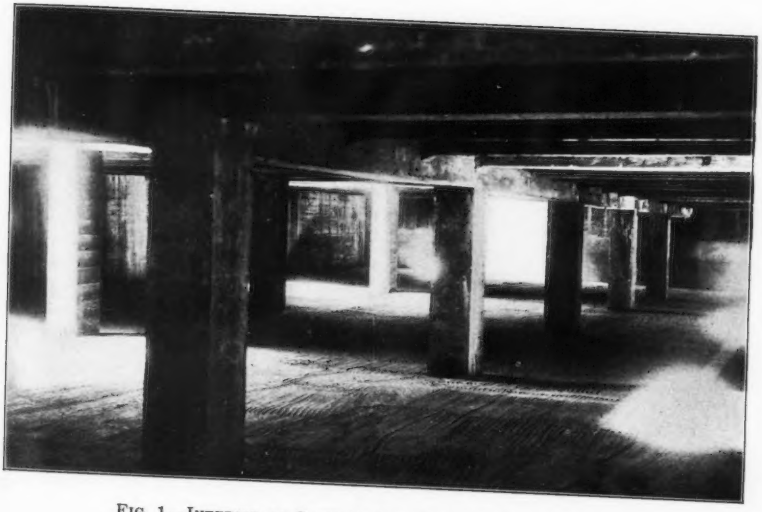

FIG. 1.—INTERIOR OF SLOW FILTER, SHOWING SAND IN PLACE.

FIG. 2.—TOPS OF SLOW FILTERS, SHOWING SAND TROUGHS.

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system of drains running over the tops of the filters and discharging to the sewer through the overflow of each slow filter. The sand boxes have covers of ¼-in. checkered steel plates stiffened by angle irons riveted to the underside thereof, the cover-plates being in two halves for convenience in handling. Manhole heads and covers, 18 in. in diameter, are provided for the compartments containing the weirs, and 24-in. heads and covers for the manhole opening into the filter, except for the main entrance, for which there is a rectangular checkered steel cover, 2 by 4 ft. The sand boxes have 2-in. drain-tile connections at the bottom of each, and lead into the 4-in. tile drains between two adjacent rows of sand boxes; these connections allow the water to drain out of the sand when the box is filled from the sand-washing plant.

Frost Proofing.—Around the periphery of the plant a curb, 2.5 ft. high, of ornamental design was built on top of the roofs of the filters, and the entire roof, after the pipe drains were laid, was covered with earth, Fig. 1, Plate LIII, to a depth of 2.5 ft., the upper 9 in. being of top soil. The whole area was then raked over and sowed with grass seed.

Exterior Finish.—The visible exterior faces of the slow filter walls were divided into panels by pilasters moulded monolithic with the walls, the effect being carried around the roughing filters as a water-table from which to start the brickwork of the building containing the regulating rooms, storage rooms, etc.

Supply and Drain Valves.—In the southwest corner of each of the three slow filters is a compartment, Plate LII, the top edge of which is level with the surface of the sand. The rough-filtered water is delivered to the filters through these compartments. Drains for emptying the water from the surface of the filters, and overflow pipes connecting with a 12-in. drain, lead to the sewer. The drains from the sand troughs on the roof discharge vertically downward into the overflow pipe.

Slow-Filter Underdrains.—The underdrainage system of the slow filters consists, Plate LII, and Fig. 2, Plate LIV, of lines of 10-in. half-tiles lying at right angles to the length of the filter in the center of each panel between the rows of posts, and connecting with a main underdrain running lengthwise of the filter in one of the panels adjacent to the center line; a 12-in. cast-iron pipe leads from

this central drain to the regulating house. The effluent control valves for all three filters are placed in a special house forming a part of the structure standing on the roughing filters. The half-tile drains were extended to a point about 4.5 ft. from the walls, all around, the ends being closed, and where they cross the central underdrain they were covered with concrete slabs forming part of the tight cover of the main underdrain. The vitrified hub and spigot tile drains were laid with open joints, and were surrounded with layers of broken stone, the first or bottom layer, about 6 in. thick, being composed of fragments of crushed sandstone the largest pieces of which were not more than 3 in. in diameter and the smallest not less than ¾ in. in diameter; the second layer, 3 in. thick, was formed of sandstone particles from ¼ to ¾ in. in diameter, from which the dust had been removed and in which there were few particles larger than ½ in. in diameter; and the top layer, also 3 in. thick, was prepared from sandstone screenings, from which the dust had been removed and in which there were no particles larger than ½ in. in diameter and very few larger than ¼ in. in diameter. The material was secured from a quarry at Marysville, on the Susquehanna, some miles above Harrisburg, and was in three sizes, ¾ in. to 3 in., ¼ to ¾ in., and screenings, the latter containing a considerable quantity of fine dust as well as particles up to ¾ in. in diameter. By proper sorting the only material requiring a second handling was the screenings, which, on sifting through ¼-in. and ¾-in. sieves, provided for all the fine material and for nearly one-half the ¼ to ¾-in. gravel required; but the weight of the wasted dust from the screenings exceeded, by approximately 25%, the weight of the useful materials secured. As received at the works, the materials were quite dirty, and a simple plan for washing out the stone dust was devised by the contractor's foreman. This consisted of a couple of oil barrels mounted tandem on horizontal axes slightly above the centers of the barrels, on a framework high enough to allow a dump-cart to stand thereunder. The two barrels were provided with a simple lever arrangement for turning them upside down. After being filled with water from a hydrant, they were suddenly upset on a cart containing, perhaps, ½ cu. yd. of broken stone. This quick flush effectively washed the mud from the whole depth of stones in the cart, the surplus water draining out under the tailboard.

The gravel underdrains were not carried out to the side walls of the

FIG. 1.—SLOW-FILTER OPERATING ROOM.

FIG. 2.—SAND WASHER.

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filter, but were stopped about 2 ft. therefrom and banked up to the proper height, the finer layers of gravel covering the coarser layers on the slopes, and the top surface being brought to uniform level.

Filter Sand.—The filter sand was secured from the Susquehanna River, and required no further preparation than screening, and, at times, the mixing of sands obtained from different localities.

The specifications for the sand were as follows:

Not more than 10% should be smaller than 0.2[illegible] mm. in diameter.
Not more than 5% should be smaller than 0.24 mm. in diameter.
At least 90% should be finer than 0.80 mm. in diameter.

Practically all the particles should be finer than 1 mm. in diameter, and the uniformity coefficient should be not greater than 1.6 to 1.8.

These specifications were written for the Susquehanna River sand with which the writer had had much experience; a modification of these requirements might be desirable for sands from other localities.

As to chemical composition, the Susquehanna River sands were satisfactory, containing less than 2% of lime and magnesia, and less than 5% of an aggregate amount of iron and aluminum in forms likely to disintegrate under the action of water or chemical changes produced by matters the water might contain when passing through the sand. The analyses of the samples of sand taken after the filters were ready to be put in operation are given in Table 1.

Filter No. 3 and practically all of Filter No. 2 were filled with sand obtained at Northumberland, from the West Branch of the Susquehanna, and required no further preparation than the careful selection of the barge loads as the sand was dredged, accepting those which would fill the requirements satisfactorily, as indicated by field tests conducted as the dredging proceeded, and rejecting those in which the sand was too fine or too dirty. The only difficulty experienced with this sand was the likelihood of its containing noticeable quantities of saw-dust, shreds of bark, and friable particles, but casting it by hand over screens having ¼-in. meshes removed the objectionable matters to a degree which rendered the sand satisfactory for filtration purposes.

Owing to the frequent changes in the conditions of flow in the Susquehanna, the character of the sand deposited is subject to variation, and, before a sufficient quantity had been secured, the Northumberland deposits had become too fine for use and it was necessary

to seek a coarser sand to mix with it in order to increase its effective size. An inspection of several deposits farther up the river finally disclosed one at Nesbit, from which much of the finer material had been washed, and by combining the mechanical analyses of this and the Northumberland sands, it was found that a satisfactory composite sand could be made by mixing the two in proper proportions.

TABLE 1.—MECHANICAL ANALYSES OF SAND IN SLOW FILTERS OF STEELTON FILTER PLANT.

Note.—In this table the figures in Column 1 give the depth from which the sample was taken, those in Column 2 refer to samples collected on the northeast corner of the filter, those in Column 3 to samples of the southeast corner, those in Column 4 to samples collected in the center of the filter, those in Column 5 to samples collected in the northwest corner of the filter, and those in Column 6 to samples collected in the southwest corner of the filter.

(1)	(2)		(3)		(4)		(5)		(6)	
	NORTHEAST CORNER.		SOUTHEAST CORNER.		CENTER.		NORTHWEST CORNER.		SOUTHWEST CORNER.	
Depth from which sample was taken.	Effective Size.	Uniformity Coefficient.	Effective Size.	Uniformity Coefficient.	Effective Size.	Uniformity Coefficient.	Effective Size.	Uniformity Coefficient.	Effective Size.	Uniformity Coefficient.
FILTER NO. 1.										
Top 0.5 ft.	0.32	1.7	0.32	1.5	0.32	1.7	0.31	1.5	0.31	2.0
0.5 to 1.5 ft.	0.30	1.8	0.30	1.6	0.31	1.7	0.31	1.6	0.31	1.9
1.5 to 3.0 ft.	0.29	1.9	0.32	1.5	0.32	1.5	0.30	1.6	0.32	2.0
3.0 to 4.0 ft.	0.31	1.5	0.33	1.3	0.29	1.7	0.31	1.6	0.32	1.8
FILTER NO. 2.										
Top 0.5 ft.	0.32	1.5	0.30	1.5	0.30	1.4	0.31	1.4	0.30	1.4
0.5 to 1.5 ft.	0.32	1.5	0.30	1.5	0.30	1.4	0.30	1.5	0.30	1.4
1.5 to 3.0 ft.	0.30	1.4	0.31	1.4	0.30	1.5	0.31	1.4	0.30	1.4
3.0 to 4.0 ft.	0.31	1.4	0.40	1.3	0.31	1.4	0.30	1.4	0.29	1.5
FILTER NO. 3.										
Top 0.5 ft.	0.31	1.6	0.30	1.8	0.31	1.8	0.31	1.7	Samples not taken.	
0.5 to 1.5 ft.	0.32	1.6	0.30	1.8	0.33	1.7	0.29	1.8		
1.5 to 3.0 ft.	0.32	1.6	0.29	1.7	0.36	1.5	0.30	1.8		
3.0 to 4.0 ft.	0.31	1.7	0.32	1.7	0.36	1.5	0.28	1.8		

As an example, Table 2 exhibits the analyses of the Northumberland and Nesbit sands on July 2d, 1908, at which time the mixing of the two sands in equal proportions produced the desired composite.

TABLE 2.

Sieve: Meshes, per inch.	Size of separation of sieves, in millimeters.	Nesbit sand. Percentage passing sieve.	Northumberland sand. Percentage passing sieve.	Composite sand. Percentage passing sieve.
20	1.02	100	100	100
30	0.57	64	65	65
50	0.38	8	23	13
60	0.29	2	14	7
80	0.21	0.3	0.6	0.5
Effective size		0.4 mm.	0.27 mm.	0.32 mm.
Uniformity coefficient		1.4	2.00	1.6

When Filter No. 1 was being filled, the Northumberland sand had an effective size of only about 0.27 or 0.28 mm. The two sands were mixed by dumping cart loads of each kind upon separate platforms on each side of a manhole, and casting simultaneously the correct relative numbers of shovelsful from each pile upon a screen standing over the manhole. In passing through the screen and falling to the filter below, the two sands became thoroughly mixed, as will be seen from an inspection of the analyses in Table 1.

The sand was delivered to the filters in one-horse, two-wheeled carts. It was dumped through the manholes, and spread in three layers, care being taken not to compact it unevenly. The top surface, Fig. 1, Plate LIV, was leveled off with straight-edges to a practically uniform elevation, the final operation being to rake the entire area over lightly with an iron rake. A typical mechanical analysis of the Northumberland sand, when running at its best, is given in Table 3.

The Northumberland sand, during the process of dredging from the river bottom, was quite thoroughly washed, and required no further treatment except screening through ¼-in. mesh screens to remove mussel shells, shreds of bark, etc., before being placed in the filters.

The total quantity of sand shipped to Steelton for the slow filters was 3 989 tons, in 107 car loads, and made, in the filters, 2 953 cu. yd.

In order to help out the contractor in the matter of securing this

sand, the writer kept an inspector at the sand dredges, whenever sand was being secured, during the entire period covering November and December, 1907, and January, April, May, June, and July, 1908; actual dredging operations, however, were limited to 92 days during this time.

TABLE 3.—MECHANICAL ANALYSIS OF SAMPLES OF NORTHUMBERLAND SAND, OCTOBER 31ST, 1907.

Size of sieve. Meshes, per linear inch.	Effective size of separation, in millimeters.	Weight of sand passing sieve, in pounds.	Percentage passing.
4	3.4	2.295	100
10	2.15	2.289	99.7
20	1.02	2.274	99
30	0.57	2.183	95
40	0.45	1.774	76
50	0.38	1.067	46
60	0.29	0.170	7.4
80	0.21	0.025	1.1
100	0.17	0.012	0.6
150	0.12	0.005	0.0

Note.—Effective size, 0.31 mm.; uniformity coefficient, 1.37: finer than 0.24 mm., 2.5%; finer than 1 mm., 99%; finer than 1.5 mm., 99.5 per cent.

The total cost of inspecting the sand, including the salary of the inspector, his board, and traveling expenses, was 9.5 cents per ton, or 13.0 cents per cu. yd. As the sand was running, a satisfactory indication of its character was obtained by a mechanical analysis of one sample to about each 30 cu. yd. dredged. The sand was sampled on the barge before unloading, each sample being a composite of samples collected at about four places from freshly excavated vertical faces in the sand as piled on the barge, and collected so as to be representative of the material for the full depth of the pile. These samples were then thoroughly mixed, the sample for analysis being taken therefrom, dried, and sifted through carefully rated screens.

Sand obtained in the river deposits in the neighborhood of Harrisburg could not have been used without expensive preparatory treatment to remove the fine material, as its effective size usually runs from about 0.23 to 0.24 mm. To remove some of the fine material, washing is necessary, the total waste ranging from about 10% to more than 50%, depending largely on the quantity of coal dust in the sand. The sand from Northumberland, being obtained from the West Branch above the junction of the North Branch, is free from coal, and is also coarser than that obtained farther down the river.

Effluent Control of Slow Filters.—The effluent pipes pass beneath the floors of the slow filters, and each terminates in its own regulating chamber with a sluice-gate controlled from a valve-stand and hand-wheel on the floor of the operating room. The rate of filtration of each filter is controlled by causing the filtered water to flow through a submerged orifice, Fig. 4, 8 in. in diameter in the end of an outlet pipe having a swivel-joint at the lower end, Plates XLVII and XLVIII, and discharging through a gate-valve into a filtered-water well common to the three filters. The free or orifice end of the swivel-

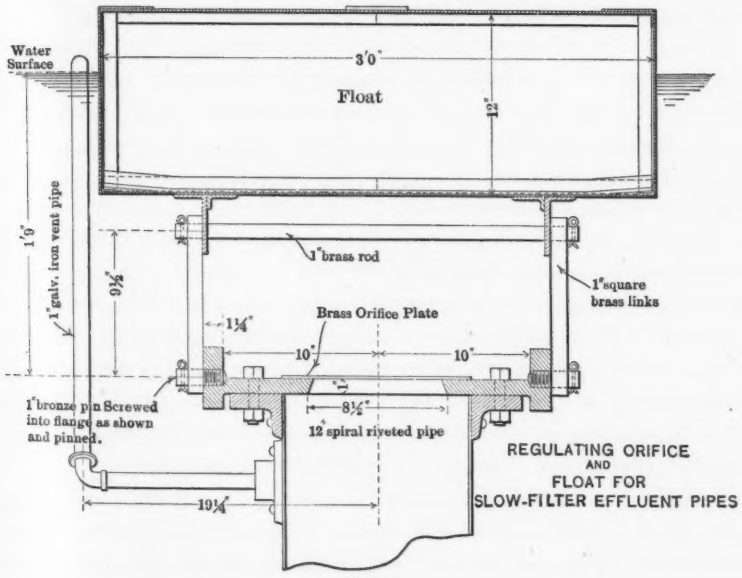

FIG. 4.

jointed pipe is suspended by brass links from a float resting on the surface of the filtered water in the effluent chamber of the filter; the center of the 8-in. orifice is adjusted to stand 1 ft. 9 in. below the surface of the water in the effluent well, at which depth, with a free discharge through the orifice into the atmosphere, the filter will deliver 1 500 000 gal. per day to the outer well. If the pumps in the Borough pumping station do not take the water away as fast as the three filters can furnish it, the water rises in the outer well, submerges the orifices, and cuts down the head by an amount just sufficient to keep up the supply, the plant being capable, therefore, of yielding the water to the

pumping station automatically at any rate necessary, up to the maximum for which the plant is designed, namely, 1 500 000 gal. per day per filter. In order to prevent any draft-tube effect in the effluent pipe, between the effluent well and the outer filtered-water well, a 2-in. galvanized-iron pipe is connected with the swivel-jointed pipe at a point below the orifice, the free end of the 2-in. pipe being adjusted so as to be always above the water level in the effluent well, whatever position the float may take; this assures a discharge into the atmosphere whenever the difference in level of the water between the effluent well and the filtered-water well of any filter exceeds 1 ft. 9 in.

Gauge Boards.—The rate of filtration and loss of head are indicated on scale boards, Fig. 1, Plate LV, carried by bronze wires attached to floats in the different compartments of the regulating house, as described for the roughing filters. A sampling pump, similar to those for the roughing filters and the raw water, is provided for the effluent well of each slow filter.

The filtered water leaves the central filtered-water well through a 24-in. wood-stave pipe leading to the new pump-well at the pumping station, a sluice-gate being provided at each end of this pipe line.

Sand Washer.—The system of handling the sand when the slow filters require cleaning is not new in principle, but the details differ materially from those of other plants. Portable ejectors, supplied with water at about 100-lb. pressure from a line of 4-in. wrought-iron pipe, Plate LII and Fig. 2, Plate LIII, suspended from the ceiling of each slow filter, through 3-in. bronze hose-gates and 3-in. hose, lift the dirty sand and transport it to the sand-washer plant. The dirty sand, having been shoveled into piles in the filter, is cast into the hopper of the portable ejector, is picked up by the jet of water passing through the ejector, and delivered through a line of 4-in. wrought-iron pipe to the sand-washing hoppers, Plate XLVIII, and Fig. 2, Plate LV, in the second story of the slow-filter regulating house. After passing through two hoppers the washed sand is forced by a jet in the bottom of the second hopper through a line of 4-in. wrought-iron pipe, resting on the roof of the three filters, to hose-valves from which lines of hose may be laid to discharge the returning clean sand into the compartments of the sand troughs running lengthwise of the tops of the filters. The operation is simple, and requires no special description or comment.

Heating.—The building covering the operating rooms of the rough-

ing and slow filters, as well as the machinery room and the coagulant-storage and sand-washer rooms, is heated by steam generated in a vertical tubular boiler standing in the second story of the slow-filter regulating house. The returns from the radiators are piped back to a trap in one corner of the slow-filter operating room, and discharge into the sewer. The provisions for heating required boiler capacity with sufficient heating surface and grate area to furnish steam to raise the temperature of 20 000 lb. of coagulant solution from 32° to 70° Fahr., in 1 hour. The boiler is designed to carry 100 lb. of steam, and is provided with a reducing valve, on the steam-heating connection, set at 5 lb. The coils and radiators are proportioned to maintain the machinery and operating rooms at a temperature of 70° and the coagulant-storage room and the sand-washer room at a temperature of 58° when the outdoor temperature is at zero. The toilet-room is provided with a wash-basin with cold-water connection and a steam connection for securing hot water, and a low-down tank closet.

Construction.

No special difficulties were encountered during construction, the bearing value of the foundations proving satisfactory for the relatively light loads they are required to carry. There was comparatively little ground-water in any of the work, although the permanent ground-water level was barely below the floor of the excavation for the slow filters; in fact, a modification of the design of the main underdrains of two of the filters was made necessary in order to keep the work out of the water. A considerable portion of the site of the slow filters was, in early days, a low swale which had been used subsequently as a dumping ground for slag, cinders, and ashes from the steel works, and it was owing to the presence of these materials, which had had such a long time to become thoroughly settled, that satisfactory foundations were secured without extra work.

Leakage.—Before the sand was placed in the slow filters they were filled with water from the street mains up to the level required for operation, the water from the street being then shut off and the drop in the surface level being observed. It was not expected that the filters would be perfectly tight, owing to the construction of the bottoms in blocks about 11 ft. square, with no special provision for stopping leakage between the blocks. The leakage from each of the

three filters was about the same, and aggregated about 0.66 of 1% of the nominal daily capacity of the plant. No tests to determine the amount of leakage have been made since the filters have been in operation, but it is believed that there has not been an increase.

Cost of the Plant.—The plans were completed in July, 1907, and, in response to advertisements, three bids were opened on August 7th, 1907, aggregating $71 522.85, $80 482.48, and $89 592.59, respectively. The contract, which was awarded August 14th, 1907, to the Bunting Construction Company, of Flushing, N. Y., stipulated that the work should be completed within 6 months from the date of the contract, or by February 14th, 1908. Various delays resulted, however, and it was not until the middle of September that the plant could be put in operation, the final estimate to the contractor being dated September 24th, 1908, and amounting to $70 730.87. This, however, did not represent the full cost of the work, as the installation of the centrifugal pumps, the changing of the suction pipes at the pumping station, and some repair work on the checkered steel covers on the slow filters, were done by the Water Board outside of the contract. In addition to the above items, there were the cost of the site and of engineering and incidental expenses, which were high, by reason of the fact that the time occupied by the contractor in the construction of the plant was more than twice that in which the work should have been completed under favorable conditions.

The following are the principal items making up the cost of the plant:

Excavation, grading, roadways, sodding, etc..	$3 936.88
Sewers and drains	2 314.67
Concrete, all classes	18 708.32
Reinforcing steel	3 475.80
Manhole heads and covers, indoors and outdoors	1 590.50
Cast-iron pipes and specials	3 956.27
Wood-stave pipes between filter plant and pumping station	9 034.21
Filter regulating devices, gauges, coagulant and lime-water apparatus	2 650.00
Superstructure over regulating houses, etc., including steam heating, lighting, and plumbing	7 900.00

Machinery, plant, motors, compressor, shafting, belting, air receiver, sand-washing plant, etc.	3 750.00
Gate and sluice valves	1 485.00
Filtering materials, including underdrains	10 811.34
Extra work	1 117.88
	$70 730.87

Of the above amount, $57 000.00 is the cost of the purification works, the remainder, $13 700.00, being the cost of the pipes to the pumping station, the sewer to the river and other details growing out of local conditions.

Operation.

The plant was put in service on September 12th, 1908, but was shut down later in the day, and was started again on September 16th, since which date it has furnished filtered water continuously.

The regular force required for operation consists of a general superintendent and two filter attendants, the latter working on 12-hour shifts. When the slow filters require scraping an additional force is taken on temporarily, the men working under the direction of the general superintendent.

The permit for the construction of the plant, issued by the Pennsylvania State Department of Health, provided that the filters should be operated for a considerable period under the direction of its designer, and in compliance with this condition the writer instructed the superintendent and attendants in their duties, provided specific instructions regarding the manipulation of the roughing and slow filters, and the handling of the coagulant, and has advised with reference to special features of the operation whenever necessary.

From September 16th, 1908, until January 9th, 1909, no coagulant was used in connection with the operation of the roughing filters, the raw water being sufficiently free from turbidity to yield a satisfactory water without coagulant.

Quantity of Coagulant Required.—The general instructions given to the superintendent called for the application of a coagulant to the raw water whenever its turbidity exceeded 50 parts per million, and the continuance of its use for much lower turbidities whenever the

last bacterial analyses showed 5 000 or more bacteria per cu. cm. in the raw water, or in case the effluent from the slow filters showed objectionable color. It is the intention, in dosing with coagulant, to use a sufficient quantity to produce an effluent from the roughing filters having a turbidity of not more than 10% of that of the applied water, and not, in any case, more than 25 parts per million, regardless of the turbidity of the raw water. It has also been found desirable, particularly during the winter and the cold spring weather, to manipulate the dose of coagulant so as to produce an effluent from the rough-

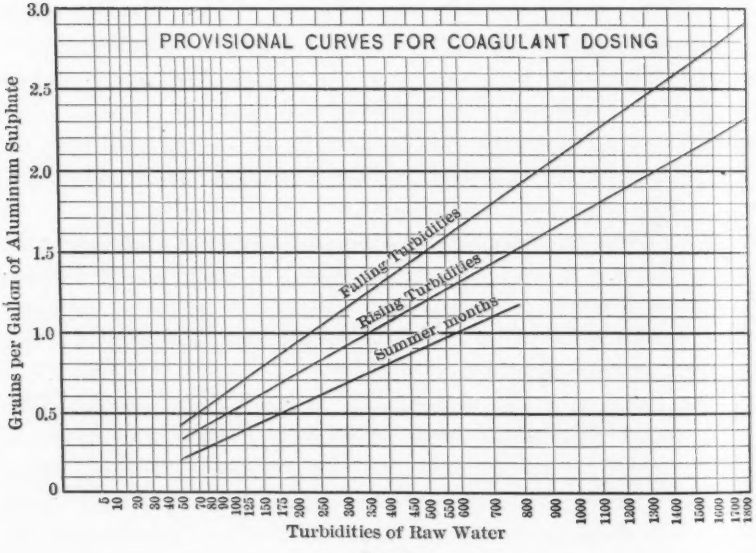

FIG. 5.

ing filters having a turbidity of practically zero, as by this means the runs of the slow filters are very greatly lengthened and the cost of operation is not increased by the use of the comparatively small additional amount of coagulant necessary to secure this result.

The quantity of coagulant to be used is determined by the superintendent from a diagram, Fig. 5, prepared by the writer from data secured originally during the operation of the Harrisburg Testing Station, in 1902 and 1903, but modified somewhat to take into account the difference in the character of the filtering materials, and the slightly different character of the raw water at the Steelton intake; the

diagram may require further modification as additional knowledge is gained during the operation of the plant. The dose of coagulant indicated by the lowest curve is for application during the summer when the turbidities do not rise very high, or very suddenly. The dosing indicated by the middle curve is required when the turbidity is rising rapidly, during the first parts of the floods in the fall, winter and spring; the dosing indicated by the upper curve is required when the turbidity is falling, after the passage of a flood. It will be noticed that for equal turbidities more coagulant is required when the turbidities are falling than when they are rising, on account of the finer character of the turbidity and the persistence of high numbers of bacteria in the Susquehanna after the turbidities have begun to fall. The superintendent must use judgment in determining the proper dose of coagulant. The character of the turbidity of the river water varies so greatly and changes so quickly that a dose satisfactory with a given clay turbidity, say, of 500 parts per million, might prove altogether too much for an equal turbidity from the North Branch. At times, also, particularly after a protracted season of low water, a turbidity caused largely by the fine shreds of vegetation torn and scoured loose from the rocky river bottom is particularly difficult to handle by reason of its high clogging value when it becomes matted upon the filter surface. In order to keep the plant in efficient operation, therefore, the superintendent must watch the effect of his dosing and reduce the quantities if he finds the loss of head increasing on the roughing filters too rapidly, or increase the quantity if the conditions require it.

When to Wash the Roughing Filters.—If, when coagulant is being applied to the raw water in the proper amount, the effluent of a roughing filter shows objectionable turbidity, indicating the passage of coagulated material through the filter bed, the filter is immediately washed; otherwise, it is left in operation until the maximum allowed loss of head, about 2.5 ft., is reached.

This method of operation is based on the theory, which seems to be supported by considerable evidence gained both at this plant and at the Harrisburg Testing Station, that these coarse-grained filters act very much as settling basins with bottom areas of great extent. The mud carried into the filter by the entering water is deposited on the granules of the filter bed through practically its entire depth, and one

of the requirements in controlling their operation is to prevent the washing of this sediment through the filters by pushing their operation too rapidly, or by using too great losses of head, or by suddenly increasing the rates of filtration. To prevent most of these evils the mechanisms at the plant limit the loss of head to about 2.5 ft.

Regulation of Coagulant Dosing.—The quantity of solution required to supply a given dose of coagulant depends on the quantity of water being filtered and the percentage strength of the solution.

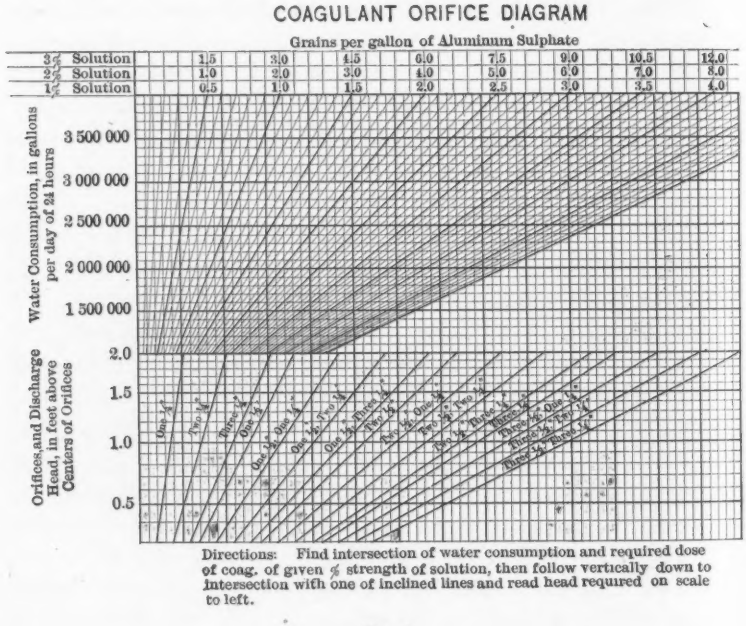

FIG. 6.

Knowing these, the requisite quantity of solution is measured out by continuous discharge through orifices in the glass plate forming the front of the measuring box, the size and number of orifices, as well as the head required to give the necessary dose, in grains per gallon, being ascertained by the superintendent from a diagram, Fig. 6, prepared for his use. The lines indicating the orifices to be used were placed on the diagram in accordance with the measured discharge of each orifice.

Use of Lime-Water.—During floods the alkalinity of the river water

falls sometimes too low to permit of the decomposition of the requisite dose of aluminum sulphate, and provisions are made to add, at such times, sufficient milk of lime to supply the deficiency in alkalinity. When lime must be used, enough is added to neutralize the free CO_2 in the river water, combine with the aluminum sulphate, and leave a residual alkalinity in the water of about 6 parts per million. The free CO_2 and alkalinity are determined by the methods recommended by the Committee on Standard Methods of Water Analysis, of the American Public Health Association. Knowing the dose of aluminum sulphate being used, and having ascertained the alkalinity of the raw water, and the amount of free CO_2 carried, reference to a diagram including all these data will give the quantity of lime required, if any.

Regulation of Lime Dosing.—The quantity of water required to add this dose of lime to the raw water in a saturated solution is regulated at the lime-water measuring box in the manner in which the coagulant dose is regulated, a diagram similar to the coagulant-dosing diagram being supplied for the lime-water. The amount of lime to be slaked daily to produce the required quantity of lime-water, for different dosings and different daily consumptions of water, is determined by the superintendent from another diagram, small quantities being slaked from time to time during the day and discharged as milk of lime through the funnel and pipe leading to the bottom of the lime-mixing tanks. The strength of the lime-water solution is to be tested from time to time by the usual simple chemical test, and it should be kept up to full strength by the attendant.

Another diagram gives the number of pounds of aluminum sulphate required for making solutions of different percentage strength, the capacity of the tanks having been calibrated for the purpose.

Simplicity of the Dosing System.—No calculations being required on the part of the superintendent during the operation of the plant, the rapid changing of the dosing, when required, can be effected with accuracy and without the likelihood of errors from faulty calculations.

The lime-water and the coagulant solution are admitted to the water practically together. The conditions requiring the use of lime being comparatively rare no great pains have been taken to go into refinements in the use of lime that would be desirable under conditions requiring its more continuous use.

Reduction of Alkalinity by Coagulant.—As to the proportion of the coagulant rendered inactive through its absorption by suspended matters in the water, sufficient data have not yet been obtained at the Steelton plant to speak authoritatively, but it is believed, from present indications, that the reduction of alkalinity of the raw water, instead of the theoretical 8.2, will probably be about 6 parts per million, per grain of coagulant used, as observed at the Harrisburg Filter Plant during the past three years and at the Harrisburg Testing Station in 1902 and 1903.

Operation of Roughing Filters.

Operation with Respect to Lengths of Runs.—While the roughing filters must be operated in a manner to secure an effluent sufficiently free from turbidity and bacteria to be handled satisfactorily by the slow filters, their operation must be controlled in a manner to secure sufficiently long periods of operation between washings, otherwise the plant could not be kept going during periods of very turbid water. A little experience is required to enable this to be done, but it presents no practical difficulty with any range of conditions thus far encountered on the Susquehanna at Harrisburg and Steelton. The highest turbidity experienced since the plant has been in operation occurred in the forenoon of January 26th, reaching 1 600 parts per million, remaining at that figure for about 5 hours, and dropping by steps to 500 parts during the next 12 hours. The shortest run during this period was about 2½ hours, which occurred after the turbidity had dropped to about 700 parts. The first turbid water the plant was required to handle occurred about January 8th, at a time when the writer was not able to be present, and, owing to the deterioration of certain of the silica-turbidity standards, the turbidities of the raw water and of the effluents from the roughing filters were incorrectly read, the actual turbidities being much higher than the figures recorded. As a result, the quantity of coagulant used was entirely too small, and a considerable number of bacteria passed through both the roughing filters and the final filters.

Records of Operation.—Since January 23d, 1909, with but very few exceptions, the coagulant has been properly applied, and the satisfactory results obtained in the removal of turbidity are exhibited in Table 4, which gives the hourly records of the operation of Roughing Filter No. 1, through runs Nos. 58 to 81, January 26th to 31st, 1909, inclusive.

TABLE 4.—Hourly Records of Operation of Roughing Filter No. 1 Through Runs Nos. 58 to 81, January 26th to 31st, 1909.

Number of run.	Lengths of run.		Date.		Turbidities, in parts per million.			Coagulant used, in grains per gallon.
	Hours.	Minutes.	Day.	Hour.	Raw water.	Rough-filtered water.	Percentage of removal.	
58	9	58	Jan. 26	6 A.M.	80	1	98.8	0.42
				7 "	800	1	99.9	0.42
				8 "	800	1	99.9	0.92
59	4	16		9 "	800	1	99.9	1.84
				10 "	1 600	1	99.9	2.22
				11 "	1 600	1	99.9	2.22
				12 M.	1 600	1	99.9	2.22
				1 P.M.	1 600	1	99.9	2.22
60	3	02		2 "	1 600	1	99.9	2.22
				3 "	900	1	99.9	2.00
				4 "	900	1	99.9	2.00
61	2	31		5 "	900	1	99.9	2.00
				6 "	900	1	99.9	2.00
				7 "	800	1	99.9	2.00
62	2	26		8 "	700	1	99.8	1.50
				9 "	700	1	99.8	1.50
63	4	35		10 "	700	1	99.8	1.50
				11 "	700	1	99.8	1.00
				12 "	500	1	99.8	0.75
			Jan. 27	1 A.M.	500	1	99.8	0.75
				2 "	500	2	99.6	0.75
64	2	58		3 "	500	2	99.6	0.75
				4 "	500	2	99.6	0.75
				5 "	500	1	99.8	0.75
65	2	11		6 "	500	1	99.8	0.75
				7 "	600	2	99.7	0.63
				8 "	750	2	99.7	0.63
66	3	18		9 "	800	3	99.6	0.63
				10 "	800	2	99.8	0.63
				11 "	750	2	99.7	0.63
				12 M.	750	2	99.7	0.63
67	3	17		1 P.M.	700	2	99.7	0.63
				2 "	600	3	99.5	0.63
				3 "	550	2	99.6	0.63
68	4	15		4 "	550	2	99.6	0.63
				5 "	550	2	99.6	0.63
				6 "	550	3	99.4	0.63
				7 "	550	3	99.4	0.63
69	3	53		8 "	550	3	99.4	0.63
				9 "	550	3	99.4	0.63
				10 "	550	3	99.4	0.80
				11 "	500	10	98.0	1.95
				12 "	500	2	99.6	1.95
70	3	00	Jan. 28	1 A.M.	500	1	99.8	1.50
				2 "	500	1	99.8	1.50
				3 "	500	1	99.8	1.50
71	4	03		4 "	500	1	99.8	1.50
				5 "	500	1	99.8	1.50
				6 "	500	1	99.8	1.50
				7 "	500	1	99.8	1.30

TABLE 4.—(*Continued.*)

Number of run.	Lengths of run.		Date.		Turbidities, in parts per million.			Coagulant used, in grains per gallon.
	Hours.	Minutes.	Day.	Hour.	Raw water.	Rough-filtered water.	Percentage of removal.	
72	4	04	Jan. 28.	8 A. M.	500	1	99.8	1.30
				9 "	500	1	99.8	1.30
				10 "	500	1	99.8	1.30
				11 "	500	1	99.8	1.22
73	4	56		12 M.	450	1	99.7	1.22
				1 P. M.	500	1	99.8	1.22
				2 "	500	1	99.8	1.22
				3 "	500	1	99.8	1.22
				4 "	500	1	99.8	1.22
				5 "	500	1	99.8	1.22
74	5	13		6 "	500	1	99.8	1.22
				7 "	500	1	99.8	1.30
				8 "	450	1	99.7	1.30
				9 "	450	1	99.7	1.30
				10 "	450	1	99.7	1.30
75	5	32		11 "	450	1	99.7	1.30
				12 "	450	1	99.7	1.30
			Jan. 29	1 A.M.	450	1	99.7	1.30
				2 "	450	1	99.7	1.30
				3 "	450	1	99.7	1.40
				4 "	450	1	99.7	1.40
76	5	19		5 "	450	1	99.7	1.40
				6 "	450	1	99.7	1.40
				7 "	400	1	99.7	1.40
				8 "	400	1	99.7	1.40
				9 "	300	1	99.7	1.40
77	6	50		10 "	300	0	100	1.30
				11 "	300	0	100	1.30
				12 M.	300	0	100	1.30
				1 P.M.	300	0	100	1.30
				2 "	275	0	100	1.30
				3 "	275	0	100	1.20
				4 "	275	0	100	1.20
				5 "	275	0	100	1.20
78	8	05		6 "	250	0	100	1.12
				7 "	250	0	100	1.12
				8 "	250	0	100	1.12
				9 "	250	0	100	1.12
				10 "	250	0	100	1.12
				11 "	250	0	100	1.12
				12 "	250	0	100	1.12
			Jan. 30	1 A.M.	250	0	100	1.12
79	10	59		2 "	250	0	100	1.12
				3 "	250	0	100	1.12
				4 "	250	0	100	1.12
				5 "	250	0	100	1.12
				6 "	250	0	100	1.12
				7 "	250	0	100	1.12
				8 "	250	0	100	1.12
				9 "	250	0	100	1.12
				10 "	250	0	100	1.12
				11 "	250	0	100	1.12
				12 M.	250	0	100	1.12
80	17	00		1 P.M.	200	0	100	1.12
				2 "	200	0	100	1.12
				3 "	200	0	100	0.99

TABLE 4.—(*Continued.*)

Number of run.	Lengths of run.		Date.		Turbidities, in parts per million.			Coagulant used, in grains per gallon.
	Hours.	Minutes.	Day.	Hour.	Raw water.	Rough-filtered water.	Percentage of removal.	
80	17	00	Jan. 30	4 P. M.	200	0	100	0.99
				5 "	200	0	100	0.99
				6 "	200	0	100	0.99
				7 "	200	0	100	0.99
				8 "	200	0	100	0.99
				9 "	200	0	100	0.99
				10 "	200	0	100	0.99
				11 "	200	0	100	0.99
				12 "	200	0	100	0.99
				1 A.M.	200	0	100	0.99
				2 "	200	0	100	0.99
				3 "	200	0	100	0.99
				4 "	150	0	100	0.99
				5 "	150	0	100	0.99
81	13	40		6 "	150	0	100	0.99
				7 "	150	0	100	0.99
				8 "	100	0	100	0.63
				9 "	100	0	100	0.63
				10 "	100	0	100	0.63
				11 "	100	0	100	0.63
				12 M.	75	0	100	0.55
				1 P.M.	75	0	100	0.55
				2 "	75	0	100	0.55

Four of the runs included in Table 4 were too short, and, with the experience since gained in handling the plant, particularly the coagulant, could have been extended by an hour or two without difficulty.

The complete records of the operation of Roughing Filter No. 1 by runs, from September 12th, 1908, to February 3d, 1909, are given in Table 5.

Analyses.—There is no laboratory for chemical and bacteriological work at the plant, but an arrangement has been made by which samples of the raw water, the rough-filtered water, the final-filtered water, and water secured from a faucet in the borough, are collected and examined by the chemist and bacteriologist of the Harrisburg Filter Plant once each week, and at such other times as desired. Two plates, for total numbers of bacteria, are prepared from each sample, and presumptive tests are made for the presence of *B. Coli* in the raw- and filtered-water samples, five 1-cu. cm. sowings usually being made of each sample; the tests for turbidity, free CO_2, and alkalinity are made at the plant by the superintendent, who is a skilled chemist.

TABLE 5.—(*Continued.*)

Run No.	From:	To:	Time in operation: Hours.	Time in operation: Minutes.	Loss of head: Initial.	Loss of head: Final.	Average rate of operation, in gallons daily.	Millions of gallons per acre per day.	Silica turbidity, in parts per million: Raw water.	Silica turbidity, in parts per million: Effluent.	Silica turbidity, in parts per million: Percentage removed.	Bacteria per cubic centimeter: Raw water.	Bacteria per cubic centimeter: Effluent.	Bacteria per cubic centimeter: Percentage removed.	Coagulant, in grains per gallon (average).
36	Jan. 10	Jan. 11	6	02	0.50	1.60	780 000	84.3	400	19.0	95	.....	....	..	....
37	" 11	" 11	4	21	0.50	1.58	890 000	102.0	400	20.0	95	18 200	6 000	68	0
38	" 11	" 11	5	18	0.48	1.95	810 000	92.9	400	28.0	93	.....	....	..	0
39	" 11	" 11	4	00	0.52	1.96	810 000	92.9	400	30.0	92	.....	....	..	0
40	" 11	" 11	3	11	0.55	1.95	880 000	100.8	400	30.0	92	.....	....	..	0.8
41	" 11	" 12	2	58	0.83	1.65	860 000	98.5	450	19.0	96	.....	....	..	0.9
42	" 12	" 12	3	40	0.60	2.15	750 000	84.9	450	13.0	97	.....	....	..	0.9
43	" 12	" 12	4	09	0.45	2.15	800 000	91.7	500	9.0	98	.....	....	..	0.9
44	" 12	" 12	2	44	0.60	1.55	890 000	102.0	500	6.0	98	.....	....	..	0.9
45	" 12	" 12	2	38	0.65	1.15	820 000	94.0	500	5.0	99	.....	....	..	0.9
46	" 12	" 12	3	43	0.50	1.90	830 000	95.0	500	6.0	98	.....	....	..	0.9
47	" 12	" 12	4	02	0.60	1.87	720 000	82.5	500	9.0	98	.....	....	..	0.9
48	" 12	" 13	8	01	0.74	1.55	770 000	88.2	450	7.0	98	.....	....	..	0.9
49	" 13	" 14	27	57	0.45	1.55	760 000	87.0	450	24.0	95	.....	....	..	0.3
50	" 14	" 15	12	05	0.52	1.74	770 000	88.2	300	5.0	98	.....	....	..	0.40
51	" 15	" 15	9	01	0.60	1.60	790 000	90.7	150	4.0	97	32 000	6 200	80	0.38
52	" 15	" 15	9	45	0.45	1.75	790 000	90.7	150	4.0	97	.....	....	..	0.33
53	" 15	" 16	4	40	0.55	1.22	760 000	87.1	125	4.0	97	.....	....	..	0.25
54	" 16	" 20	83	39	0.57	1.46	750 000	86.0	50	4.0	92	.....	....	..	....
55	" 20	" 23	68	14	0.48	1.60	720 000	82.5	20	3.0	85	5 975	1 445	76	0.10
56	" 23	" 24	17	38	0.62	1.45	730 000	83.7	15	1.0	93	.....	....	..	0.25
57	" 24	" 25	19	13	0.50	1.29	660 000	75.7	40	2.0	95	.....	....	..	0.25
58	" 25	" 26	9	58	0.38	1.75	630 000	72.2	560	1.0	99	.....	....	..	0.45
59	" 26	" 26	4	16	0.40	1.60	680 000	78.0	1 440	1.0	99	.....	....	..	2.07
60	" 26	" 26	3	02	0.40	2.10	780 000	89.4	1 200	1.0	99	.....	....	..	2.08
61	" 26	" 26	2	31	0.95	2.25	1 000 000	114.7	900	1.0	99	.....	....	..	2.00
62	" 26	" 26	2	26	0.95	1.96	840 000	96.2	700	1.0	99	.....	....	..	1.50
63	" 26	" 27	4	35	0.54	2.25	650 000	75.7	600	1.0	99	.....	....	..	1.58
64	" 27	" 27	2	58	0.57	1.98	860 000	98.5	500	2.0	99	.....	....	..	0.75
65	" 27	" 27	2	11	0.45	2.32	910 000	104.3	650	2.0	99	13 160	1 025	92	0.67
66	" 27	" 27	3	18	0.44	2.20	750 000	86.1	750	2.0	99	.....	....	..	0.63
67	" 27	" 27	3	17	0.40	1.75	810 000	92.9	600	2.0	99	.....	....	..	0.63
68	" 27	" 27	4	15	0.40	1.89	760 000	87.1	550	3.0	99	.....	....	..	0.63
69	" 27	" 28	3	53	0.45	2.20	830 000	95.2	525	4.0	99	.....	....	..	1.19

TABLE 5.—SUMMARY OF RESULTS OF OPERATION OF ROUGHING FILTER NO. 1, BY RUNS.

Run No.	From:	To:	Time in operation: Hours.	Time in operation: Minutes.	Loss of head: Initial.	Loss of head: Final.	Average rate of operation, in gallons daily.	Millions of gallons per acre per day.	Silica turbidity, in parts per million. Raw water.	Silica turbidity, in parts per million. Effluent.	Silica turbidity, in parts per million. Percentage removed.	Bacteria per cubic centimeter. Raw water.	Bacteria per cubic centimeter. Effluent.	Bacteria per cubic centimeter. Percentage removed.	Coagulant, in grains per gallon (average).
1	Sept. 12	Oct. 5	570	43	0.35	1.76	710 000	81.4	6	....	...	.....	....	..	....
2	Oct. 5	" 19	334	00	0.53	1.18	707 000	81.0	2	0.5	75	.....	....	..	....
3	" 19	" 29	245	46	0.45	1.98	692 000	79.3	2	0.3	87	.....	....	..	....
4	" 29	Nov. 3	125	03	0.45	1.70	700 000	80.3	7	0.0	100	.....	....	..	....
5	Nov. 3	" 8	125	37	0.53	1.18	690 000	79.0	3	0.0	100	80	....	..	....
6	" 8	" 12	90	07	0.45	0.75	720 000	82.5	2	0.0	100	.....	....	..	....
7	" 12	" 16	95	23	0.54	0.85	685 000	78.5	2	0.0	100	.....	....	..	....
8	" 16	" 20	91	46	0.55	1.04	760 000	87.1	2	0.0	100	119	76	39	....
9	" 20	" 28	186	41	0.51	1.32	700 000	80.3	2	0.0	100	214	12	92	....
10	" 28	Dec. 4	161	00	0.45	1.68	770 000	88.2	1	0.0	100	.....	....	..	....
11	Dec. 4	" 8	102	55	0.60	1.08	770 000	88.2	1	0.0	100	408	310	24	....
12	" 8	" 14	136	31	0.88	1.06	800 000	91.7	2	0.0	100	220	125	43	....
13	" 14	" 18	94	41	0.58	1.04	710 000	81.3	5	1.0	80	1 680	680	60	....
14	" 18	" 20	50	10	0.40	1.60	690 000	79.0	7	1.0	87	.....	....	..	....
15	" 20	" 22	51	09	0.38	1.50	705 000	80.8	9	3.0	66	4 425	800	80	....
16	" 22	" 24	39	02	0.60	1.61	750 000	86.0	15	3.0	80	.....	....	..	....
17	" 24	" 28	94	56	0.65	1.30	760 000	67.1	7	3.0	60	.....	....	..	....
18	" 28	Jan. 1	87	17	0.56	0.92	750 000	86.0	4	1.0	75	.....	....	..	....
19	Jan. 1	" 3	53	54	0.75	1.48	750 000	86.0	3	1.0	66	5 800	1 125	80	....
20	" 3	" 6	76	52	0.62	1.58	790 000	90.6	4	1.0	75	.....	....	..	....
21	" 6	" 7	10	16	0.58	1.04	840 000	96.3	10	4.0	60	.....	....	..	....
22	" 7	" 7	13	38	0.42	1.02	810 000	92.9	100	4.0	96	.....	....	..	....
23	" 7	" 8	6	45	0.60	1.30	870 000	99.7	225	8.0	96	.....	....	..	....
24	" 8	" 8	5	39	0.48	1.30	860 000	98.6	300	8.0	97	.....	....	..	....
25	" 8	" 8	9	57	0.48	1.56	760 000	87.1	350	16.0	96	.....	....	..	....
26	" 8	" 9	7	56	0.62	1.55	830 000	95.1	400	14.0	97	.....	....	..	....
27	" 9	" 9	4	05	0.55	1.10	850 000	97.4	425	19.0	95	.....	....	..	....
28	" 9	" 9	3	32	0.57	1.45	860 000	98.6	450	21.0	93	.....	....	..	1.3
29	" 9	" 9	4	22	0.60	1.53	840 000	96.3	500	3.0	99	.....	....	..	0.6
30	" 9	" 9	3	57	0.67	1.63	870 000	99.7	550	3.0	99	.....	....	..	0.35
31	" 9	" 10	3	37	0.53	1.42	850 000	97.4	450	6.0	98	.....	....	..	0.3
32	" 10	" 10	5	07	0.54	1.80	810 000	92.9	450	15.0	96	.....	....	..	0.3
33	" 10	" 10	4	57	0.35	1.50	850 000	97.4	400	33.0	92	.....	....	..	0.3
34	" 10	" 10	3	51	0.56	1.75	890 000	102.0	400	13.0	97	.....	....	..	0.3
35	" 10	" 10	5	14	0.54	1.70	800 000	91.7	400	18.0	96	.....	....	..	0.3

TABLE 5.—(*Continued.*)

TABLE 5.—(*Continued.*)

Run No.	From:	To:	Time in operation:		Loss of head:		Average rate of operation, in gallons daily.	Millions of gallons per acre per day.	Silica turbidity, in parts per million.			Bacteria per cubic centimeter.			Coagulant, in grains per gallon (average).
			Hours.	Minutes.	Initial.	Final.			Raw water.	Effluent.	Percentage removed.	Raw water.	Effluent.	Percentage removed.	
70	Jan. 28	Jan. 28	3	00	0.00	2.17	780 000	87.1	500	1.0	99	.....	....	..	1.50
71	" 28	" 28	4	03	0.40	2.90	760 000	87.1	500	1.0	99	.....	....	..	1.45
72	" 28	" 28	4	04	0.45	2.90	760 000	89.5	500	1.0	99	.....	....	..	1.28
73	" 28	" 28	4	56	0.60	2.16	760 000	89.5	500	1.0	99	.....	....	..	1.92
74	" 28	" 28	5	13	0.50	1.82	750 000	86.1	450	1.0	99	.....	....	..	1.28
75	" 28	" 29	3	32	0.40	2.11	730 000	82.6	450	1.0	99	.....	....	..	1.33
76	" 29	" 29	5	19	0.57	1.99	740 000	84.9	300	0.0	100	.....	....	..	1.44
77	" 29	" 29	6	50	0.57	1.99	700 000	80.3	250	0.0	100	.....	....	..	1.26
78	" 29	" 30	6	05	0.40	1.88	710 000	81.4	250	0.0	100	.....	....	..	1.12
79	" 29	" 30	8	59	0.44	1.95	580 000	66.6	250	0.0	100	.....	....	..	1.12
80	" 30	" 31	10	00	0.44	1.86	560 000	64.2	250	0.0	100	.....	....	..	1.00
81	" 30	" 31	17	40	0.65	1.43	740 000	84.9	100	0.0	100	.....	....	..	0.65
82	" 31	Feb. 1	13	38	0.55	1.40	650 000	74.5	75	0.0	100	.....	....	..	0.55
83	" 31	" 2	10	13	0.50	1.50	780 000	88.7	60	0.0	100	.....	....	..	0.50
84	Feb. 1	" 2	23	47	0.60	1.86	810 000	93.0	50	0.0	100	.....	....	..	0.50
85	" 2	" 3	12	11	0.60	1.46	790 000	90.7	30	2.0	93	.....	....	..	....

The area of the filter = 0.008719 acre.

Air Wash.—The roughing filters, when clogged, or when not yielding a satisfactory effluent, are cleaned by washing with air and water, first shutting off the raw-water supply and then, if pushed for time, opening the sewer gate and wasting the water upon the surface of the filter down to the level of the wash-water troughs. The filter, in the meantime, being in operation, is allowed to run until the surface of the water on it has dropped about 6 in. below the edges of the wash-water troughs; the effluent gate is then closed, and compressed air is discharged into the underdrains from the air receiver. The compressed air is stored in the receiver at a pressure of approximately 100 lb. per sq. in. at the beginning of the wash, and is discharged through a reducing valve set at about 10 lb. on the filter side of the valve; this, however, does not represent the pressure at which the air is applied during the wash, as the 10 lb. is largely used up in friction through the small air pipes leading to the filters. The air issues into the filter through the orifices in the bottom of the 2-in. galvanized-iron underdrain pipes 1 in. above the floor of the filter. The bottoms of the pipes lying all in the same plane, the air issues from the orifices only as it is forced out by the pressure in the delivery pipe, its tendency to issue by its own buoyancy being overcome by placing the orifices in the bottoms of the pipes; within limits, therefore, it is possible to deliver to the filter any quantity of air desired, and to have it issue through all the orifices in the underdrain pipes with practical uniformity. The application of the air is continued until the contents of the receiver, approximating 1 500 cu. ft. of free air, is discharged, which takes from 4 to 5 min., as the reducing valve is now set, and this corresponds to approximately 1 cu. ft. of free air per square foot of filter surface per minute. Apparently, this is sufficient, for, as far as can be ascertained, there has been no tendency for suspended matters to accumulate in the filter permanently. The objection to the use of a large quantity of air, or to very heavy air washing, is the likelihood of the disturbance of the gravel underdrains to be followed by the loss of some of the filtering material; and the tendency to aid in producing vertical stratification in the filter, in connection with the water-wash, allowing some portions to clog up completely, and increasing the velocity of filtration through the more porous parts to such an extent as to render the filter ineffective. In order to guard against this tendency, the superintendent, from time to time, stirs up by hand the

filtering materials in the roughing filters, using a rake while the wash-water is being applied; this dissipates effectively any tendency of the materials to collect in vertical strata, and maintains the porosity and efficiency of the filter.

Water-Wash.—After the air is turned off the filters are washed with filtered water from the street mains. The water is applied through the underdrains, and the dirty water overflows into the wash-water troughs and passes thence to the river through the sewer. The wash-water is ordinarily applied at a rate equivalent to about three and one-half times the rate of filtration, or between 8 and 9 vertical inches in the filter per minute, the duration of the water-wash being from 5 to 6 min., and the quantity of water used per wash about 10 000 gal. In addition to that used for washing, about 5 000 gal. are wasted from the surface of the filter at each wash when the turbidity is high and the filters have to be washed frequently. It cannot, as yet, be told with accuracy just what percentage of wash-water will be required during the year, but a prediction of the quantity can be made from data secured at the Harrisburg Testing Station, and at the Harrisburg Filter Plant. The number of washes required will depend on the turbidity of the raw water, as this causes the clogging of the filter and necessitates the use of a coagulant.

TABLE 6.—EFFECT OF TURBIDITY OF RAW WATER ON LENGTHS OF RUNS OF ROUGHING FILTERS AT STEELTON.

TURBIDITY OF APPLIED WATER, IN PARTS PER MILLION.		Number of days per year when such turbidities may be expected to prevail.	Average lengths of runs.
Range. (1)	Average. (2)	(3)	(4)
0– 50	25	160	6 days.
51– 75	62	65	24 hours.
76– 100	88	35	17 "
101– 150	125	29	12 "
151– 200	175	23	10 "
201– 250	225	13	9 "
251– 500	375	25	7 "
501– 800	650	7	5 "
801–1 200	1 000	5	4 "
1 200+	1 400	3	3 "

Lengths of Runs.—The data thus far secured from the Steelton plant, in addition to information from the Harrisburg Testing Station, indicate that the lengths of runs from the roughing filters, when operat-

ing at a rate of from 75 000 000 to 125 000 000 gal. per acre per day, would be about as shown in Table 6. Column 1 indicates the range of turbidity of the raw water, Column 2 the average turbidity, Column 3 the approximate number of days per year when such turbidities may be expected to prevail, and Column 4 the lengths of runs of the roughing filters.

Using the data in Table 6, 428 washes per year per filter will be required, calling for a total of about 6 700 000 gal. of wash- and waste-water, assuming that the water on the surface of the roughing filters is wasted at each wash. As it takes about 15 min. to wash a filter, the necessary lost time for the 428 washes would be 4½ days per year, and the output of each filter for the 360.5 days of operation, at its full rated capacity, 1 500 000 gal. per day, would be 540 500 000 gal.; the wash- and waste-water, on this basis, would represent 1.2% of the net output of the filter for the year.

The longest run yet experienced was about 24 days, and the shortest 2 hours 11 min. It is believed that with more experience the shortest runs need not be less than about 4 hours.

TABLE 7.—AVERAGE DOSE OF COAGULANT.

TURBIDITY OF APPLIED WATER, IN PARTS PER MILLION.		Number of days per year when such turbidities may be expected to prevail. (3)	Average dose of aluminum sulphate, in grains per gallon. (4)	Average dose of lime, in grains per gallon. (5)
Range. (1)	Average. (2)			
0– 50	25	160	0.0	0
51– 75	62	65	0.4	0
76– 100	88	35	0.5	0
101– 150	125	29	0.6	0
151– 200	175	23	0.7	0
201– 250	225	13	0.8	0
251– 500	375	25	1.0	0.25
501– 800	650	7	1.4	0.40
801–1 200	1 000	5	1.7	0.60
1 200+	1 400	3	2.1	0.75

Average Dose of Coagulant.—From the data thus far secured, it would appear that the average quantities of coagulant necessary in the operation of the plant would be about as shown in Table 7, from which a calculation indicates that the average dose required through a series of years would be equivalent to 0.37 gr. per gal., or 53 lb. per million gallons of water treated, which, at $20 per ton, gives the average cost for coagulant as 53 cents per million gallons. For the

first year the quantity required will fall far short of this figure, the average dose from September 16th, 1908, to April 18th, 1909, having been but 0.22 gr. per gal.

Effect of Turbidity on Initial Loss of Head.—The initial loss of head, at the commencement of operation of a roughing filter, is not affected either by the turbidity of the raw water or the quantity of aluminum sulphate used, the effects of these two factors being evident only in the rate at which the loss of head increases, and consequently in the lengths of the runs.

Effect of Rate on Quantity Filtered.—The rate of filtration affects only slightly the total quantity of water that may be filtered between washings of the roughing filters; whatever slight effect there may be apparently indicates that somewhat larger quantities can be filtered between washings at high rates than at low rates, but, of course, at the expense of somewhat more frequent washings.

Operation of Slow Filters.

The slow filters are operated on the continuous plan, the rough-filtered water being controlled so as to stand about 2 ft. deep on the surface of the slow filters.

The method of regulating the rate of filtration need not be again described, it being sufficient to state that the yield of the slow filters is responsive to the draft of the municipal high-pressure pumps, up to the limit of the discharging capacity of the regulating devices on the filters. As filtration proceeds, the slow filters gradually clog up, and the loss of head increases; provision is made to allow for a total loss of head of 7 ft. in the slow filters, which is the full depth of the filtering materials, and the superincumbent water.

Daily Records.—The slow filters were put in operation on September 16th, 1908. The daily records of the operation of Filter No. 1 are given in Table 8, from which it will be seen that the accumulation of the loss of head proceeded with comparative slowness during September, October, and November, while during December the occasional increase in turbidity of the raw water began to have its effect on clogging, bringing the total loss of head up to about 2 ft. on the last day of December. In January, 1909, during which there were two periods of bad water, the loss of head increased to about 6 ft., a gain of 4 ft., during the month, and it became necessary to start scraping.

On the morning of January 30th, therefore, the supply to Filter No. 1 was shut off, the water was drained down to the surface of the filter, and filtration was continued for a short time to allow the water to fall a sufficient distance below the sand surface.

TABLE 8.—RECORD OF OPERATION OF SLOW FILTER No. 1, SEPTEMBER 16TH, 1908, TO FEBRUARY 1ST, 1909.

Date.	Time in service: Days.	Hours.	Loss of head, in feet.	Rate of Operation: Gallons daily.	Millions of gallons per acre per day.
(1)	(2)		(3)	(4)	(5)
Sept. 16	..	..	0.38	680 000	4.7
" 17	1	..	0.36	650 000	4.5
Oct. 1	15	..	0.33	610 000	4.2
" 4	18	..	0.35	600 000	4.1
" 9	23	..	0.45	600 000	4.1
" 10	24	..	0.30	520 000	3.6
" 14	28	..	0.49	510 000	3.5
" 17	31	..	0.35	500 000	3.4
" 23	37	..	0.40	530 000	3.7
" 28	42	..	0.35	630 000	4.3
" 31	45	..	0.30	520 000	3.6
Nov. 4	49	..	0.40	580 000	4.0
" 14	59	..	0.63	500 000	3.4
" 25	70	..	0.61	500 000	3.4
" 30	75	..	0.75	500 000	3.4
Dec. 1	76	..	0.75	500 000	3.4
" 5	80	..	0.85	500 000	3.4
" 8	83	..	1.70	500 000	3.4
" 11	86	..	1.50	680 000	4.7
" 17	92	..	1.20	500 000	3.4
" 23	98	..	1.60	500 000	3.4
" 26	101	..	1.33	500 000	3.4
" 27	102	..	1.95	500 000	3.4
" 31	106	..	2.07	500 000	3.4
Jan. 1	107	..	2.09	500 000	3.4
" 3	109	..	2.20	500 000	3.4
" 4	110	..	2.35	500 000	3.4
" 5	111	..	2.42	500 000	3.4
" 9	115	..	2.92	500 000	3.4
" 12	118	..	3.70	500 000	3.4
" 15	121	..	3.74	500 000	3.4
" 18	124	..	4.23	500 000	3.4
" 22	128	..	4.12	500 000	3.4
" 25	131	..	5.01	500 000	3.4
" 27	133	..	5.35	500 000	3.4
" 28	134	..	5.99	500 000	3.4
" 29	135	..	5.75	500 000	3.4
" 30	135	7	6.01	500 000	3.4

Filter put in service September 16th, 1908.
Filter scraped January 30th, 1909.
Sand removed, washed, and stored, 18 cu. yd. = 124 cu. yd. per acre.
Quantity filtered during run = 71 000 000 gal.
Millions of gallons per acre filtered between scrapings = 491.
Average rate of filtration = 3 623 000 gal. per acre daily.
Net area of sand surface = 0.1446 acre.
Wasted from surface of filter for scraping .....104 500 gal.
Water drained out, and water used for refilling from below...156 800

261 300 = 0.36% of filtrate.

Scraping.—Filter No. 1 was scraped by the superintendent of the plant and a force of five men, the actual time occupied in piling up

the scrapings ready for transportation to the washer being 3 hours, or at the rate of about 7 sq. ft. of filter scraped per man per minute. The scraping on this filter was rather deep, averaging nearly an inch, and being less than ¾ in. in very few places. A portion of the extra scraping was made necessary by the accidental dropping of one of the manhole covers into the filter early in January. This caused a great disturbance of the sand, over an area of perhaps 100 sq. ft., as the cover turned when it struck the water and penetrated the sand bed on its edge, the lower corner having reached almost to the gravel underdrains. The cover was taken out, and all the discolored sand which had been driven down into the bed from the *schmützdecke* was dug out and sent to the washer.

Sand Transportation and Washing.—The ejector and washing plant worked satisfactorily as to the handling of the sand, but, as it had not been in use before, the men did not attain a high degree of efficiency in its operation. It is believed that greater familiarity with its use, and the proper regulation of the relative force of the different jets, will permit of handling the sand much more rapidly and efficiently than was done. The total quantity of sand removed from Filter No. 1 was about 18 cu. yd., or 124 cu. yd. per acre of filter surface, to transport which to the sand washer, wash it, and return it to the sand troughs on top of the filters occupied about 8 hours, one man attending to the sand washer, one to the hose returning the sand to the sand troughs, one raking the scraped surface of the filter, and an average of three shoveling the dirty sand to the portable ejector.

After the sand for one filter had all been washed, the portable ejector was taken apart and an irregularly-shaped stone, which had carelessly been left in one of the water mains, was found wedged in the nozzle. This had caused the deflection of the jet of water so that the throat on the discharge side had been cut deeply. After the throat had been replaced with a new one, the original capacity of the ejector was restored, and the scraping and transporting of sand to the washer from the other filters proceeded with more expedition.

Ejector Nozzles.—The nozzle in the portable ejector throws a ⅝-in. jet of water into a throat with a 1½-in opening reduced to 1 in. in diameter 1½ in. from the front of the throat and increasing to 1⅝ in. in diameter at the back end of the throat 7 in. from the face; the nozzle and throat stand 1½ in. apart. The nozzle reduces from 2 in. in

diameter to the $\frac{5}{8}$-in. jet in a length of about 6 in. The nozzles and throats are of chilled cast iron.

The nozzles for the sand washer have a $\frac{19}{32}$-in. jet, and the throats reduce from $1\frac{1}{2}$ in. to $1\frac{1}{8}$ in. in diameter in $1\frac{1}{2}$ in., increasing to $1\frac{3}{4}$ in. in $5\frac{1}{2}$ in. The portable ejector and the washing hoppers were supplied by the Norwood Engineering Company, from designs by Allen Hazen, M. Am. Soc. C. E., originally worked out for the filter plant at Washington, D. C., and containing recent improvements suggested by Mr. Hazen. The washing plant consists of only two hoppers, and practically all the washing is done in the first one, the sand as it is thrown to the second hopper from the first being sufficiently clean to be used in the filter. The wash-water overflows the second hopper with very little turbidity.

The lift from the portable ejector to the top of the pipe discharging into the washing hopper is about 15 ft.; from the second washing hopper to the sand troughs there is a drop of about 6 ft., so that comparatively little pressure is required to transport the sand from the sand washer back to the sand bins.

The washing hoppers are provided with auxiliary jets at the bottom to compensate for the quantity of dirty water actually picked up by the jet and forced through to the succeeding hopper, the auxiliary jet also serving the purpose of stirring up the sand at the bottom of the hopper so as to facilitate its transportation by the water jet. The dirty water overflowing the washing hoppers passes through a reinforced concrete box in order to catch such sand as may escape with the wash-water; the dirty water escapes from this box to the sewer by an overflow.

The water which transports the washed sand to the sand troughs overflows a weir at one end of each sand box and escapes to the sewer.

Refilling after Scraping.—After the dirty sand is all thrown out of the filter and the surface of the clean sand has been raked over to remove footprints and give a smooth even surface, filtered water is admitted through the underdrains until the surface of the filter sand is covered about 2 in. in depth; the raw water is then admitted, gently at first, and the filter is refilled to its proper operating depth with the rough-filtered water.

Resumption of Filtration.—In starting this filter in operation after scraping, the effluent control valve was regulated so that the filter would

deliver, according to its rate-gauge, 800 000 gal. a day, it being necessary to choke down the discharge of the clean filter to a proper rate because the two other filters were very much clogged up and could not yield their proportional part of the water without a loss of head of 5 or 6 ft.; with such a loss of head the clean filter would at once start off with its maximum allowable rate, which was deemed inadvisable, the filter being so new, and having been started in operation so late in the season.

Lengths of Runs.—Filter No. 2 was scraped on February 2d and was put back in service on February 3d, and Filter No. 3 on February 4th, 1909. The lengths of runs of these three filters, therefore, were 135.5, 137.5 and 140.5 days, respectively. Filter No. 1 delivered during its run 71 000 000 gal. at an average rate of 3 623 000 gal. per acre daily, the total yield corresponding to 491 000 000 gal. per acre between scrapings. The figures for the other two filters were a trifle larger.

The total quantity of water wasted from the surface of the filter prior to scraping, and the quantity used for refilling the filter from below, before starting in operation again, corresponded to 0.14 of 1% of the quantity of water filtered.

The three slow filters have now (May 10th, 1909) been in service for 14 weeks since the last (and only) scraping, and the loss of head is but 0.7 ft.

Efficiency of the Plant.

The efficiency of the entire plant, in the removal of turbidity and bacteria, is exhibited in the daily records given in Table 9, which date from November 1st, 1908, for the reason that no bacterial analyses were made prior to November 5th.

The efficiency in the removal of turbidity has been 100 per cent. During November and December, when the turbidity of the raw water was low and the bacteria were not numerous, the effluent water was satisfactory. The same is true during January, with the exception of the second week when, as has already been explained, the superintendent was deceived by the deterioration of his silica-turbidity standards, and the quantity of coagulant was inadequate to produce a satisfactory effluent. The results subsequent to that date are good. The samples collected on the afternoon of February 5th, when Filter No. 3 was being scraped, represent the bacterial contents of the effluents of Filters Nos. 1 and 2, which had been scraped, No. 2 within two days

TABLE 9.—Daily Efficiency of Plant in Removal of Turbidity and Bacteria.

Date.	Average turbidities, in parts per million:				Bacteria, per cubic centimeter:				Average dose of coagulant, in grains per gallon.	Alkalinity of raw water, in parts per million.
	Raw water.	Rough-filtered water.	Filtered water.	Percentage of removal.	Raw water.	Rough-filtered water.	Filtered water.	Percentage of removal.		
1908.										
Nov. 1...	5	0	0	100	.....	.....	.....	.....	.....	.....
2...	4	0	0	100	.....	.....	.....	.....	.....	.....
3...	3	0	0	100	.....	.....	.....	.....	.....	.....
4...	2	0	0	100	.....	.....	.....	.....	.....	.....
5...	3	0	0	100	80	.....	3	96	.....	.....
6...	3	0	0	100	.....	.....	.....	.....	.....	.....
7...	3	0	0	100	.....	.....	.....	.....	.....	.....
8...	3	0	0	100	.....	.....	.....	.....	.....	.....
9...	2	0	0	100	.....	.....	.....	.....	.....	.....
10...	2	0	0	100	.....	.....	.....	.....	.....	.....
11...	2	0	0	100	.....	.....	.....	.....	.....	.....
12...	2	0	0	100	.....	.....	.....	.....	.....	.....
13...	2	0	0	100	.....	.....	.....	.....	.....	.....
14...	2	0	0	100	.....	.....	.....	.....	.....	.....
15...	1	0	0	100	.....	.....	.....	.....	.....	.....
16...	2	0	0	100	.....	.....	.....	.....	.....	.....
17...	3	1	0	100	.....	.....	.....	.....	.....	.....
18...	3	1	0	100	.....	.....	.....	.....	.....	.....
19...	2	0	0	100	.....	.....	.....	.....	.....	.....
20...	2	0	0	100	119	76	6	95	.....	.....
21...	2	0	0	100	.....	.....	.....	.....	.....	.....
22...	2	0	0	100	.....	.....	.....	.....	.....	.....
23...	2	0	0	100	.....	.....	.....	.....	.....	.....
24...	2	0	0	100	.....	.....	.....	.....	.....	.....
25...	2	0	0	100	.....	.....	.....	.....	.....	.....
26...	1	0	0	100	.....	.....	.....	.....	.....	.....
27...	1	0	0	100	214	12	3	99	.....	.....
28...	1	0	0	100	.....	.....	.....	.....	.....	.....
29...	1	0	0	100	.....	.....	.....	.....	.....	.....
30...	1	0	0	100	.....	.....	.....	.....	.....	.....
Dec. 1...	1	0	0	100	.....	.....	.....	.....	.....	.....
2...	1	0	0	100	.....	.....	.....	.....	.....	.....
3...	1	0	0	100	.....	.....	.....	.....	.....	.....
4...	1	0	0	100	.....	.....	.....	.....	.....	.....
5...	1	0	0	100	.....	.....	.....	.....	.....	.....
6...	1	0	0	100	408	310	52	87.2	.....	.....
7...	1	0	0	100	.....	.....	.....	.....	.....	.....
8...	1	0	0	100	.....	.....	.....	.....	.....	.....
9...	1	0	0	100	.....	.....	.....	.....	.....	.....
10...	1	0	0	100	.....	.....	.....	.....	.....	.....
11...	1	1	0	100	.....	.....	.....	.....	.....	.....
12...	2	1	0	100	220	125	8	96.4	.....	.....
13...	3	1	0	100	.....	.....	.....	.....	.....	.....
14...	4	1	0	100	.....	.....	.....	.....	.....	.....
15...	4	1	0	100	.....	.....	.....	.....	.....	.....
16...	4	1	0	100	.....	.....	.....	.....	.....	.....
17...	4	0	0	100	1 680	680	71	95.9	.....	.....
18...	7	1	0	100	.....	.....	.....	.....	.....	.....
19...	8	1	0	100	.....	.....	.....	.....	.....	.....
20...	4	1	0	100	.....	.....	.....	.....	.....	.....
21...	5	1	0	100	4 425	800	58	98.7	.....	.....
22...	15	4	0	100	.....	.....	.....	.....	.....	.....
23...	18	2	0	100	.....	.....	.....	.....	.....	.....

TABLE 9.—(*Continued.*)

Date.	Average turbidities, in parts per million:				Bacteria, per cubic centimeter:				Average dose of coagulant, in grains per gallon.	Alkalinity of raw water, in parts per million.
	Raw water.	Rough-filtered water.	Filtered water.	Percentage of removal.	Raw water.	Rough-filtered water.	Filtered water.	Percentage of removal.		
1908.										
Dec. 24...	10	3	0	100	.....	.....	.....	.....	.....	.....
25...	10	3	0	100	.....	.....	.....	.....	.....	.....
26...	6	2	0	100	.....	.....	.....	.....	.....	.....
27...	5	1	0	100	.....	.....	.....	.....	.....	.....
28...	4	1	0	100	.....	.....	.....	.....	.....	.....
29...	3	1	0	100	.....	.....	.....	.....	.....	.....
30...	3	1	0	100	.....	.....	.....	.....	.....	.....
31...	2	1	0	100	.....	.....	.....	.....	.....	.....
1909.										
Jan. 1...	4	1	0	100	5 800	1 125	100	98.3	.....	.....
2...	2	1	0	100	.....	.....	.....	.....	.....	.....
3...	3	1	0	100	.....	.....	.....	.....	.....	.....
4...	3	1	0	100	.....	.....	.....	.....	.....	.....
5...	3	1	0	100	.....	.....	.....	.....	.....	.....
6...	7	2	0	100	.....	.....	.....	.....	.....	.....
7...	120	4	0	100	.....	.....	.....	.....	.....	.....
8...	250	10	0	100	.....	.....	.....	.....	.....	.....
9...	450	14	0	100	.....	.....	.....	.....	0.5	.....
10...	400	20	0	100	.....	.....	.....	.....	0.15	.....
11...	425	20	0	100	18 200	6 000	1 105	93.9	0.30	.....
12...	450	7	0	100	.....	.....	.....	.....	0.90	.....
13...	250	12	0	100	.....	.....	.....	.....	0.10	.....
14...	180	12	0	100	.....	.....	.....	.....	0.20	.....
15...	125	4	0	100	32 000	6 200	320	99.0	0.35	.....
16...	75	4	0	100	.....	.....	.....	.....	0.30	.....
17...	60	5	0	100	.....	.....	.....	.....	0.10	0
18...	30	4	0	100	.....	.....	.....	.....	.....	.....
19...	25	4	0	100	.....	.....	.....	.....	.....	.....
20...	20	4	0	100	.....	.....	.....	.....	.....	.....
21...	35	4	0	100	5 975	1 445	115	98.1	.....	.....
22...	15	2	0	100	.....	.....	.....	.....	0.25	.....
23...	10	1	0	100	.....	.....	.....	.....	0.25	.....
24...	20	1	0	100	.....	.....	.....	.....	0.25	18
25...	50	2	0	100	.....	.....	.....	.....	0.25	23
26...	770	1	0	100	.....	.....	.....	.....	1.38	26
27...	600	2	0	100	13 160	1 025	11	99.9	0.78	23
28...	490	1	0	100	.....	.....	.....	.....	1.32	16
29...	325	0	0	100	.....	.....	.....	.....	1.35	22
30...	225	0	0	100	.....	.....	.....	.....	1.00	20
31...	100	0	0	100	.....	.....	.....	.....	0.65	22
Feb. 1...	75	0	0	100	.....	.....	.....	.....	0.55	16
2...	60	0	0	100	.....	.....	.....	.....	0.50	18
3...	20	3	1	95	.....	.....	.....	.....	.....	14
4...	20	2	0	100	.....	.....	.....	.....	0.25	.....
5*	15	0	0	100	3 025	295	188	94	0.25	.....
6...	70	0	0	100	.....	.....	.....	.....	0.25	14
7...	45	0	0	100	.....	.....	.....	.....	0.40	17
8...	190	2	0	100	.....	.....	.....	.....	0.60	14
9...	170	0	0	100	.....	.....	.....	.....	0.60	12
10...	215	0	0	100	.....	.....	.....	.....	0.60	13
11...	170	0	0	100	.....	.....	.....	.....	0.60	12

* Filter No. 3 out of service for scraping; Filter No. 1 was scraped on January 30th. and No. 2 on February 3d.

TABLE 9.—(*Continued.*)

Date.	Average turbidities, in parts per million:				Bacteria, per cubic centimeter:				Average dose of coagulant, in grains per gallon.	Alkalinity of raw water, in parts per million.	Free CO_2 in raw water, in parts per million.	Average dose of lime used, in grains per gallon.
	Raw water.	Rough-filtered water.	Filtered water.	Percentage of removal.	Raw water.	Rough-filtered water.	Filtered water.	Percentage of removal.				
1909.												
Feb. 12..	270	0	0	100	16 000	500	47	99.7	0.65	13	....	0
13..	160	0	0	100	....	....	..	....	0.65	15	....	0
14..	80	0	0	100	....	....	..	....	0.35	16	....	0
15..	50	0	0	100	....	....	..	....	0.33	14	....	0
16..	120	0	0	100	....	....	..	....	0.50	13	....	0
17..	200	0	0	100	....	....	..	....	0.70	10	....	0
18..	310	8	0	100	....	....	..	....	0.80	6	3	0.42
19..	190	0	0	100	4 500	115	25	99.5	1.00	6.5	3.5	0.42
20..	125	2	0	100	....	....	..	....	1.20	7.5	3	0.30
21..	300	0	0	100	....	....	..	....	0.90	9	4	0.30
22..	390	0	0	100	....	....	..	....	0.80	9	....	0
23..	465	2	0	100	....	....	..	....	1.10	11	....	0
24..	380	0	0	100	....	....	..	....	1.00	11.5	....	0
25..	275	0	0	100	....	....	..	....	0.85	11.5	....	0
26..	310	0	0	100	....	....	..	....	0.80	10	....	0
27..	310	0	0	100	6 850	70	10	99.8	0.80	11	....	0
28..	260	0	0	100	....	....	..	....	0.80	11	....	0
Mch. 1..	150	0	0	100	....	....	..	....	0.65	11	....	0
2..	120	0	0	100	....	....	..	....	0.65	8	....	0
3..	100	0	0	100	....	....	..	....	0.50	7	....	0
4..	65	0	0	100	....	....	..	....	0.40	6	....	0
5..	52	1	0	100	....	....	..	....	0.40	6	....	0
6..	47	1	0	100	3 425	750	61	98.2	0.00	8	....	0
7..	45	0	0	100	....	....	..	....	0.00	10	....	0
8..	60	0	0	100	....	....	..	....	0.35	11	....	0
9..	60	0	0	100	....	....	..	....	0.35	12	....	0
10..	50	0	0	100	....	....	..	....	0.35	12	....	0
11..	50	0	0	100	....	....	..	....	0.35	11	....	0
12..	90	0	0	100	....	....	..	....	0.35	11	....	0
13..	120	0	0	100	2 325	85	10	99.6	0.40	14	....	0
14..	115	0	0	100	....	....	..	....	0.40	16	....	0
15..	100	0	0	100	....	....	..	....	0.40	14	....	0
16..	75	0	0	100	....	....	..	....	0.30	14	....	0
17..	47	0	0	100	....	....	..	....	0.30	12	....	0
18..	42	0	0	100	....	....	..	....	0.30	14	....	0
19..	45	0	0	100	....	....	..	....	0.30	14	....	0
20..	40	0	0	100	....	....	..	....	0.30	14	....	0
21..	30	0	0	100	....	....	..	....	0.25	14	....	0
22..	25	0	0	100	....	....	..	....	0.25	16	....	0
23..	22	0	0	100	1 000	55	5	99.5	0.25	16	....	0
24..	20	0	0	100	.. .	....	..	....	0.00	15	....	0
25..	20	0	0	100	....	....	..	....	0.25	15	....	0
26..	25	0	0	100	....	....	..	....	0.25	14	....	0
27..	110	0	0	100	*	220	17	....	0.40	14	....	0
28..	140	0	0	100	....	....	..	....	0.55	14	....	0
29..	120	0	0	100	....	....	..	....	0.57	12	....	0
30..	150	0	0	100	....	....	..	....	0.62	14	....	0
31..	135	0	0	100	....	....	..	....	0.57	14	....	0
Apr. 1..	100	1	0	100	....	....	..	....	0.47	12	....	0
2..	85	0	0	100	....	....	..	....	0.45	14	....	0
3..	72	1	0	100	2 950	56	16	99.5	0.40	16	....	0
4..	60	1	0	100	....	....	..	....	0.32	16	....	0
5..	50	1	0	100	....	....	..	....	0.35	17	....	0
6..	50	1	0	100	....	....	..	....	0.40	19	....	0
7..	50	1	0	100	....	....	..	....	0.40	19	....	0
8..	55	0	0	100	....	....	..	....	0.45	18	....	0
9..	100	0	0	100	....	....	..	....	0.52	15	....	0
10..	95	0	0	100	....	....	..	....	0.48	14	....	0
11..	80	0	0	100	....	....	..	....	0.45	16	....	0
12..	95	0	0	100	....	....	..	....	0.50	15	....	0
13..	80	1	0	100	1 500	38	4	99.7	0.45	14	....	0
14..	42	1	0	100	....	....	..	....	0.35	14	....	0
15..	140	1	0	100	....	....	..	....	0.49	13	....	0
16..	345	1	0	100	....	....	..	....	0.80	12	....	0
17..	350	20	0	100	8 600	475	2	99.9	1.25	12	3	0.25
18..	340	0	0	100	....	....	..	....	1.00	11	2	0.25

* Raw water sample not taken.

and No. 1 within 5 days of the date of taking the sample. The individual samples of the effluents of Nos. 1 and 2 showed the following counts:

Filter No. 1, put in service, January 31st.
Sample No. 1..... 185 (collected February 5th)
Sample No. 2..... 170 (" " ")

Average..... 176

Filter No. 2, put in service February 3d.
Sample No. 1..... 200 (collected February 5th)
Sample No. 2..... 185 (" " ")

Average..... 198

A slight improvement shows in the effluent of the older filter.

The efficiency in the removal of *B. Coli* is shown in Table 10.

TABLE 10.—PRESUMPTIVE TESTS FOR PRESENCE OF *B. Coli* IN THE RAW WATER, FILTERED WATER AT THE FILTER PLANT, AND TAP WATER IN STEELTON, PA.

Date.	Raw Water.		Filtered Water.		Tap Water.	
	Number of 1-cu. cm. sowings.	Positive indications.	Number of 1-cu. cm. sowings.	Positive indications.	Number of 1-cu. cm. sowings.	Positive indications.
Nov. 20, 1908.	5	3	5	0	5	0
" 27	5	3	5	0	5	0
Dec. 6	5	3	5	0	5	0
" 12	5	1	5	0	..	..
" 17	5	1	5	0	5	0
" 21	4	1	4	1	4	0
Jan. 1, 1909.	5	1	5	0	5	0
" 11	5	5	5	4	5	5
" 15	5	3	5	1	5	0
" 21	5	4	5	0	5	1
" 27	5	5	5	0	5	0
Feb. 5	4	3	4	0	4	0
" 12	4	4	4	0	4	0
" 19	3	1	3	0	3	0
" 27	5	5	5	1	5	0
Mch. 6	5	2	5	0	5	0
" 13	5	3	5	0	5	0
" 23	5	4	5	0	5	0
" 27	..	..	5	0	5	0
Apr. 3	5	5	5	0	5	0
" 13	4	3	4	0	4	0
" 17	4	4	4	0	4	0

The presence of *B. Coli* in the filtered water on December 21st, January 11th, and January 15th was in part due to lack of, or insufficient use of, coagulant in the preliminary process. The dropping of

a manhole cover into Filter No. 3 on January 1st, however, may have contributed to the deterioration of the effluent during the early part of that month.

Thus far, no chemical analyses have been made of the applied water and final filtrate at the Steelton plant, and no data, therefore, have been secured on the degree of nitrification accomplished, or on the other usual changes expected. It is evident that between periods of turbid water, when no coagulant is being used, the slow sand filters will be penetrated deeply by such suspended matters as may pass through the roughing filters, among which would be bacteria, and in this respect the work required of these filters will differ from that performed by the ordinary slow sand filter. During the past few months no visible suspended matters have been left in the rough-filtered water.

In nearly all plants using aluminum sulphate as a coagulant, trouble, either continuously or intermittently, has been experienced with discoloration of the water, generally only the hot water, by iron rust. This trouble has been more or less serious at Harrisburg, Pa., Watertown, N. Y., Charleston, S. C., Hackensack, N. J., and various other places. Thus far, there has been no trouble of this sort at Steelton, although at times as much as 2 gr. of aluminum sulphate has been used per gallon of water filtered.

Cost of Operation.—The plant has not yet been in operation sufficiently long to obtain accurate data, but an approximation for an average year would indicate an expense, per million gallons, when filtering 3 000 000 gal. of water daily, about as follows:

COST OF PURIFICATION OF STEELTON WATER PER MILLION GALLONS, ON A BASIS OF FILTERING 3 000 000 GAL. PER DAY.

Operation of Roughing Filters.

Labor	$1.55	
Superintendence	0.42	
Supplies and analyses of water	0.30	
Coagulant	0.53	
Power	0.08	
Wash-water	0.13	
Light	0.05	
Coal	0.10	
		$3.16

Operation of Slow Filters.

Scraping, 5 times per year	$0.60	
Transporting and washing sand	0.06	
Re-sanding filters	0.25	
Superintendence	0.42	
		$1.33
Total		$4.49

Interest and sinking fund charges are not included.

Acknowledgments.—The principal credit for the existence of the Steelton filter plant is due to J. V. W. Reynders, M. Am. Soc. C. E., President of Councils of Steelton, Chairman of the Filtration Committee of Councils and Vice-President of the Pennsylvania Steel Company, who, by arranging a well-conducted campaign of education and devoting much thought and time to the cause, prepared the way to enable Councils to see the necessity of making provisions for a purer water supply.

On its completion, the plant was turned over for operation to the Board of Water Commissioners, Mr. George H. Roberts, President, under the general direction of the Superintendent of the Water-Works, Mr. O. P. Baskin. The operation of the filters is in charge of Mr. M. B. Litch, as Filter Superintendent, the writer, acting in an advisory capacity, as required.

The works were built by contract under the supervision of the writer's engineering force, Mr. Paul Hooker being Principal Assistant during the designing of the works, and Resident Engineer during their construction; to his faithfulness the plant bears permanent witness.

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PAPERS AND DISCUSSIONS

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CHARACTERISTICS OF MODERN HYDRAULIC TURBINES.

By Chester W. Larner, Esq.

To be Presented October 20th, 1909.

The design and selection of turbines for hydraulic power-plants were at one time—and that within comparatively recent years—processes of an extremely uncertain nature. They partook quite largely of experimentation, much of which, having no sound basis of scientific knowledge, degenerated into the purest guesswork. During late years, however, the various problems involved have been the subject of much careful and intelligent study, and the situation has become decidedly better, although there is still room for vast improvement.

One of the chief obstacles in the path of progress at the present time is the scarcity of published data on the performance of turbines of recent construction. A large quantity of exceedingly valuable information has been accumulated by the manufacturers, but very little of it is available to the public in published form, or, in fact, in any form whatever. The publication of tests seems to be an idea altogether abhorrent to the majority of turbine builders in America. It is with this policy of reticence that the writer is inclined to take issue, not only because it is of an unprofessional character, but because

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

it defeats the very commercial ends which, according to some peculiar method of mistaken reasoning, it is supposed to further. It is only by a broad-minded policy of publicity on the part of the manufacturers, that the Engineering Profession at large can become acquainted with the possibilities of turbine construction, and thereby profit in the selection of this apparatus by the progress which is being made in the development of the art.

The publication of data relative to results which have been accomplished does not necessitate the publication of any information concerning the details of design on which these results depend. Any manufacturer is justified in protecting for his own use any points of superiority in design which he may be able to devise. His success depends on them, and he could not reasonably be expected to publish them for the benefit of his competitors; but the results which can be accomplished by his machines in operation are quite a different matter. This information is of vital importance to all engineers interested in the development of water-power, and it is in pursuance of this conviction that the writer has prepared this paper.

In the evolution of turbine design in America, probably no factor has played a more important part than the Holyoke testing flume. Up to the present time, more than 1 800 turbines have been tested there, and these tests have undoubtedly had a very important influence on the advancement of the art. The fact that all wheels (within certain limits of capacity)* are tested there under essentially equivalent

* The Holyoke flume is best adapted to the testing of wheels from 27 to 42 in. in diameter, approximately. At the time the flume was built, wheels were of much lower capacity, and it was not designed to handle the large volume of water discharged by high-speed wheels 45 in. in diameter or greater. The writer names 42 in. as the limit, not because he believes that it is impossible to get good results with a wheel of larger size, but because no engineer of judgment would think of installing a larger wheel to operate under similar conditions, and dependence cannot be placed on such tests. If the conditions are less favorable than would be found in an ordinary plant, they are certainly unsuitable for a test where the object is to determine the best performance of which the wheel is capable. The main defect in design which affects the tests of large wheels is the shallowness of the tail-race. The floor of the flume, on which the draft-tube of the wheel stands for test, is only 5 ft. above the flat bottom of the tail-race. The draft-tube of a 49 or 50-in. wheel, unless it is very short, will be about 7 ft. in diameter at the lower end. The back pressure due to a column of water 7 ft. in diameter impinging on the flat bottom of the tail-race, 5 ft. from the end of the draft-tube, can be easily imagined. It has a throttling effect upon the discharge which cuts down the effective head, but does not show on the gauge, and hence the efficiency, as shown by the test, suffers. The loss does not seem to be altogether proportionate to the quantity discharged, but, apparently, is affected very largely by the relation of the draft-tube diameter to the depth of the tail-race in which the column has to turn to flow toward the weir. A 7-ft. column of water cannot readily be turned through an angle of 90° in a space of 5 ft. without considerable disturbance. The writer on one occasion tested a wheel which discharged 275 cu. ft. per sec. and found that the tail-water level, as determined by pressure readings from the draft-tube, was actually 1 ft. higher than the level in the well where the head gauge floats. The head, as shown by the gauge, was about 10 ft., whereas the real head on the wheel was only about 9 ft. Naturally, it is useless to expect good efficiency under such conditions. This is an extreme case, but wheels of smaller size, down to about 42 in. in diameter, undoubtedly suffer in the same way, although to a less extent.

conditions, adds greatly to the value of the results, because it renders them directly comparable. Tests made in place are subject to the effects of so many local conditions that one scarcely knows when it is fair to make direct comparisons, but Holyoke tests are not subject to this uncertainty, because the conditions are known and constant.

Further than this, the writer believes that it is not going too far to state that the Holyoke tests are more accurate than a majority of those made elsewhere, and are as accurate as the best of them. There is no other device for measuring power which is as generally reliable as the friction brake, when accurately adjusted. At the Holyoke flume the power of all wheels is determined by friction brakes, of which there are several sizes. The water is measured, according to the Francis formula, by a sharp-crested weir of adjustable width. The width of the weir is adjusted according to the discharge of the wheel, in order to bring the head on the weir within the limits of the experiments on which the Francis formula is based. This is in accordance with the best practice elsewhere. Furthermore, the arrangements for taking simultaneous readings, and the system of testing in general, are much superior to the conditions which often obtain in the case of tests in place.

These facts, together with the unquestionable impartiality of the tests, have given them a position of generally accepted reliability. Certain imputations to the contrary have occasionally emanated from Europe, but these are best answered by the fact that, in not a few cases, European engineers who have transplanted their activities to America have failed conspicuously to attain in the Holyoke flume the good results with which they were accredited at home.

Holyoke Tests versus Tests in Operation.—It should not be thought, from the foregoing remarks, that the writer regards a Holyoke test as necessarily an exact determination of the duty which a turbine will perform after installation. It is scarcely ever that, although, by the exercise of proper judgment, it may be a reasonably reliable indication of what may be expected in regular operation.

Tests are of interest to the Engineering Profession at large for two reasons: first, in order to determine what results have been accomplished by a machine which has been built and installed; second, as evidence of what a builder has accomplished in the past, on which to base speculations as to what he can accomplish in the case of a

proposed installation. In the first case, a test in place is generally regarded as the most satisfactory, but, in the second case, the writer finds a great diversity of opinion regarding the relative merits of Holyoke tests and tests of units in regular operation. It is conceded by all, of course, that an accurate test in operation is the only exact method of determining the performance of the turbine under working conditions, and it is further conceded by most engineers that the tests which they themselves make are accurate. However, they are not so credulous in regard to the tests made by others. The apparatus used for tests at the site is usually improvised for the occasion, is handled, to a certain extent, by unpracticed hands, and the opportunities for error are many. For this reason some engineers place more confidence in Holyoke tests because they know that they are correct and that they furnish a reliable basis on which to estimate the results which may be expected after installation.

On the other hand, some assert that wheels are tested at Holyoke under ideal conditions which cannot be obtained in practice; that the test shows merely the efficiency of the runner, and a lot of other generalities, none of which has in general any sound foundation of fact. There are certain conditions under which a Holyoke test is more favorable to the wheel than a test in place, and there are other conditions under which it is decidedly the opposite. As a matter of fact, the results of any test, whether made at Holyoke or at the site, cannot be relied on to indicate the performance of a prospective installation under different conditions of head, or speed, or capacity, without very careful study of the influence which these changed conditions will have on the efficiency of the wheel. For example, to infer that, because a manufacturer has obtained certain efficiencies with a wheel developing 10 000 h.p. under 150 ft. head at 200 rev. per min., he can obtain the same results with a wheel developing 10 000 h.p. under 100 ft. head at 300 rev. per min. would be entirely unwarranted and erroneous. Such comparisons, however, are often made, sometimes to the disadvantage of the builder, and frequently to the delusion of the purchaser.

In order to elucidate this matter, the factors which affect a comparison between wheels operating under different conditions will be analyzed. For this purpose the losses of energy in the turbine may be divided as follows:

(*a*)—Hydraulic loss in the casing, from the penstock flange to the entrance to the guides,

(*b*)—Hydraulic loss in the guides and runner,

(*c*)—Hydraulic loss due to leakage around the runner,

(*d*)—Hydraulic loss in the draft-tube,

(*e*)—Mechanical loss due to the friction of the revolving parts.

In respect to loss (*a*), the advantage is usually in favor of the Holyoke test. It is ordinarily impracticable to test a runner at Holyoke in the casing in which it is to be permanently installed, and this fact must be taken into consideration. As installed in the flume, the wheel draws the water to the guide openings smoothly and with minimum disturbance, from a state of comparative rest. Naturally, the efficiency is higher under those conditions than it will be when the wheel is installed in a usually all too constricted casing in which the water approaches the guides with whirls and eddies due to obstructions and restricted areas. There is no form of casing for a turbine that can be made within the limits of reasonable proportions, except the spiral, which does not cause considerable loss of efficiency. The loss in a properly designed spiral casing is relatively small, because it accelerates the velocity gradually and delivers the water to the guides without abrupt change of direction. A test of a wheel in an open flume can be fairly applied to a turbine of the spiral-casing type with very little allowance for additional loss, but a considerable deduction should be made if the wheel is to be installed in a cylindrical casing.

Loss (*a*) is dependent largely on the velocity in the casing, and is related to the head only in that for a given velocity the percentage of loss decreases as the head increases.

Loss (*b*) is independent of the head, provided the speed is allowed to vary as $\sqrt{h}$, but is materially affected by the type of runner and the design of the guides. The results obtained with one type of runner cannot be assumed to apply to another type. This point is fully treated in the paper.

Loss (*c*) is likewise independent of the head, and is as great at Holyoke as in any other case. Both the leakage around the runner and the discharge through it vary as the $\sqrt{h}$, and hence the ratio of one to the other (or the percentage of loss) does not vary at all.

Loss (*d*), in the case of small wheels, is probably as low at Holyoke as elsewhere. With large wheels, however, this loss is excessive.*

Loss (*e*) varies with the head, decreasing as the latter increases, because the power of the wheel increases very much faster than the friction loss due to the weight of the revolving parts. There seems to be a general impression abroad that in testing wheels at Holyoke the mechanical friction is so far minimized as to be practically eliminated. It is true that effort is made to reduce the friction load, but it is not eliminated, by any means, and, although the percentage of loss is less than it would be for the same wheel running in horizontal bearings under the same head, it is more, and sometimes a great deal more, than it would be for the same wheel operating in a horizontal position under a higher head. In fact, it is necessary to minimize the friction in order to show results which will be representative of what the wheel can accomplish under heads of from 30 to 50 or 60 ft. For wheels to operate under higher heads, a Holyoke test is, almost without exception, a disadvantage to the wheel. This fact can be readily demonstrated. Consider, for example, one of the wheels referred to later, Test No. 1799. This wheel developed 112 h.p. under 17.25 ft. head. Suppose it is desired to install a pair of these wheels on a horizontal shaft under 80 ft. head. Will the efficiency be more or less than it was in the testing flume? Under this heading, of course, the influence of mechanical friction only will be considered.

This wheel when tested had four vertical guide-bearings, one ball thrust-bearing, and a stuffing-box, in contact with the shaft. The weight on the ball-bearing, including the unbalanced thrust of the runner, was about 7 000 lb. The pressure on the upper guide-bearing under the brake was 1 000 lb., due to the pull of the load. Under 80 ft. head, two of these wheels would develop

$$\frac{112 \times 2 \times \sqrt{80^3}}{\sqrt{17.25^3}} = 2\,240 \text{ h.p.},$$

which is twenty times as much power as was developed under test. The weight of the revolving parts would be about 5 000 lb.

Now, making the impossibly liberal assumption that under test the friction of the three lower guide-bearings, the stuffing-box, and the ball-bearing was absolutely nothing, there still remains the upper guide-

* See foot-note on p. 724.

bearing, which is known, beyond question, to carry a load of 1 000 lb. Since the bearings of the proposed unit will carry a weight of 5 000 lb., the friction will be about five times as great (neglecting the influence of speed, which would modify this statement slightly).

The power developed, however, is twenty times as great, and hence the percentage of energy lost is only one-quarter as much as when the wheel was tested at Holyoke. If the friction under test was 2%, then the friction in operation will be only 0.5%, or a difference of 1.5% in favor of the latter. Under yet higher heads, the wheel in operation has a continually increasing advantage from this source.

In spite of these facts, the writer has known engineers of experience to make the broad statement that friction is so completely eliminated from Holyoke tests that they may be considered to represent purely ideal results which cannot be obtained in practice; and this in reference to heads of even 200 or 300 ft. Such statements will not stand under analysis. Under high heads, exactly the reverse is true. In fact, the writer knows of no case so ideal or so favorable to high efficiency as that of a large reaction unit operating under a high head. In such a case, not only is the mechanical friction reduced to a minimum effect, but the casing usually takes the spiral form, which minimizes the casing loss. The wheel usually has individual draft-tubes, which minimize the draft-tube loss, and the design of the runner approaches the purely inward-flow Francis type, the characteristic of which is high efficiency. The design of the wheel throughout is the most efficient which can be devised.

Scope of Tests.—Tests of turbines after installation are usually made for the purpose of determining whether a specific set of conditions has been complied with satisfactorily. These tests are usually limited to observations at various gate openings under the head and speed for which the turbine was designed. Where the power is measured electrically, the difficulty of determining the efficiency of the generator at varying speeds precludes the possibility of a comprehensive test. Such tests may fulfil their function, which is to determine the performance of a guaranty, but they add very little to the knowledge of turbine characteristics in general.

It is customary practice to design a turbine to perform a certain duty under a certain head, and either to test it under that head, or else under whatever head is available, and, from the data thus

obtained, compute the performance of the wheel under the head for which it was designed. Excepting high heads, there are few installations, however, where the conditions are by any means constant. A test made at constant speed under one head, is of little value in determining the performance of the wheel at the same speed under another head. It is of even less value in determining the general characteristics of the wheel, and, as will be demonstrated later, these characteristics are of vital importance in considering its adaptability to a given service. The characteristic curves of a wheel cannot be plotted from a test at constant speed. It is essential that the speed should vary through a considerable range. In this respect, the Holyoke tests are excellent, and, but for the fact that they are so seldom published, they would furnish a vast amount of useful information.

Selection of Wheels.—The writer is of the opinion that more trouble has resulted from the misapplication of turbines to conditions for which they are not suited, than from any other of the many mistakes which have been made in the equipment of water-power plants. There are numerous instances all over the country where a wheel which would give good results if run at the proper speed, is being run at some other speed and giving correspondingly poor results. The cause of these mistakes is not far to seek. It lies chiefly in the failure to analyze the tests of these wheels and thus determine whether they are properly adapted to the existing conditions.

Comparatively few engineers realize to what an extent the performances of different types of turbines differ. The divergence is wide, and the speed selected determines largely the type of runner which must be used. In spite of this, the speed is usually determined from a consideration of generator economy alone, without regard to its effect on the economy of the turbine. Such a course is short-sighted and ill-considered, to say the least. It results in high-speed wheels being adopted where lower-speed wheels are more suitable for many reasons. Frequently, much care and ingenuity are devoted to shackling the manufacturer with rigid guaranties for apparatus which should never be used at all for the purpose intended. Premiums are even paid to secure results which, economically considered, would be expensive if obtained gratuitously.

There is a more and more insistent demand for high-speed wheels, without a proper consideration of the limited conditions to which they

FIG. 1.—WHEEL IN FLUME READY FOR A TEST.

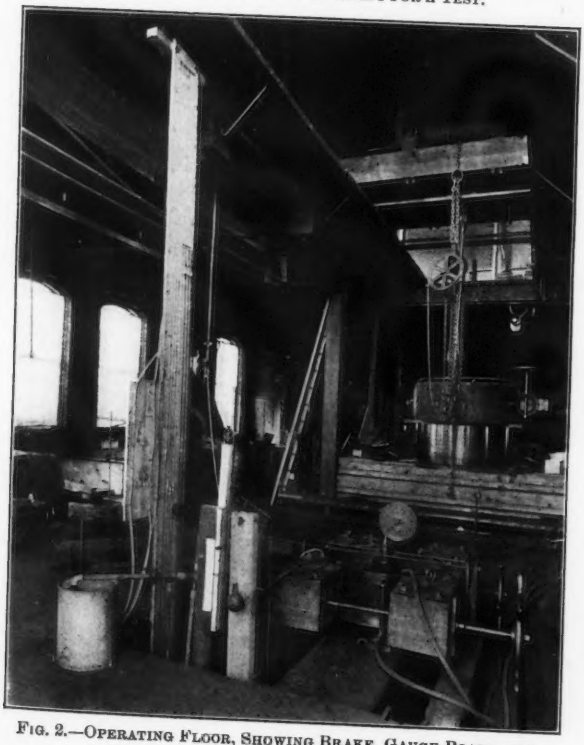

FIG. 2.—OPERATING FLOOR, SHOWING BRAKE, GAUGE-BOARD, ETC.

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are adapted and the undesirable attributes which all of them possess. The high-speed wheel, as will be explained later, has its legitimate field, but in many plants where it is being used to-day it is by no means the most economical type.

Nor has the manufacturer failed of his share of responsibility in the misuse of his wheels. Because of the long-established practice in America of standardizing runners and building lines of stock sizes, there has been too much effort directed toward adapting the plant to the wheels, rather than the wheels to the plant. This unfortunate practice has been due largely to the "cut and try" methods of the McCormick school of turbine designers, which, until recent years, dominated the turbine business in America. Each builder, having by tedious experimentation evolved his standard patterns, was accustomed to select from his stock the wheel which, according to his judgment, would most closely approximate the desired results. Frequently, the approximation was a very remote one, not more, probably, because of the rigidity of his standards than because of his meager knowledge of their characteristics. It is only within the last few years that manufacturers have learned to analyze the tests of their wheels intelligently, and to predict with accuracy what they will do under a variety of conditions. It is still more recently that designing of special wheels to suit individual requirements has come into general practice.

The writer believes that the special wheel is to be the rule in the future, rather than the exception. This should not be taken to mean that every new wheel will necessarily be an entirely new and original design. Such a procedure would be of too experimental a nature, particularly in the case of mixed-flow wheels, which are not as completely amenable to mathematical analysis as are their more simple prototypes, the Francis wheels. New designs will consist rather of modifications of and improvements on wheels which have been tested and found to be efficient, and, as such, will be more certain of success than if based wholly on theoretical considerations. The important point to be observed is that the wheel should be exactly suited to the conditions, in order that the best results may be obtained. There should be no approximation.

Description of Tests.—The purpose of this paper is not to present a study of the various problems which arise in the selection of water-wheels, but to illustrate, by means of tests, the individual charac-

teristics of several different types, and to indicate in a general way the conditions of service to which each is best adapted. The conclusions drawn from this study are based, not merely on a consideration of these tests alone, but also of all the other American and European tests which have been accessible.

The tests which have been used for illustration were made recently at the Holyoke testing flume on a series of five specially designed runners. They are reproduced from the original reports in Tables 1 to 5, inclusive. These runners range from the low-capacity Francis type wheel, No. 1795, adapted particularly to high heads, to the large-capacity high-speed wheels, Nos. 1796 and 1797, adapted particularly to low heads. The average maximum efficiency of the five wheels is 88.06 per cent. This is believed to represent the best results thus far attained in the practice of the art. Tests Nos. 1799 and 1800 are especially notable in that they show higher efficiencies than any other reliable published records in either American or European practice. Test No. 1799, showing an efficiency of 90.43% at three-quarters-gate, is probably very close to the maximum obtainable economy.

The five wheels were successively tested on a vertical shaft in the same casing. The gates were of the balanced-wicket type. All the runners were of cast iron, made in one piece from cores.

Fig. 1 is a sectional view of the arrangement for testing one of the larger wheels. The vertical thrust was carried on a ball-bearing placed on top of the crown-plate. This arrangement was necessitated by the variation in height of the runners. It was not adopted to minimize the mechanical friction, and it is not believed that the friction was any less in the case of these tests than is usual with wheels of the same size. The usual arrangement is to carry the thrust on lignum vitæ step-bearings, but the invariable practice of all manufacturers is to turn off the face of the block until a small button about 2 in. in diameter is left for bearing. While running on this button the friction is practically nothing, although it takes more of a pull to start than is required with a ball-bearing.

Fig. 2 shows the arrangement for wheel No. 1795, with an interior wooden draft-tube. This inner tube was not used with any of the other wheels.

Fig. 1, Plate LVI, is a photograph of the complete apparatus erected in the flume ready for a test. This view shows an arrangement of

TABLE 1.—TESTS OF A 32-INCH R. H. WELLMAN-SEAVER-MORGAN COMPANY TURBINE WHEEL, No. 1795.

Date, February 18th and 19th, 1909. Case No. 1794.

Wheel supported by ball-bearing step. Swing-gate. Conical draft-tube.

Number of experiment.	Proportional Part of: Percentage of full opening of speed-gate.	Proportional Part of: Percentage of full discharge of wheel.	Head acting on wheel, in feet.	Duration of experiment, in minutes.	Revolutions of wheel per minute.	Quantity of water discharged by wheel, in cubic feet per second.	Horse-power developed by wheel.	Percentage of efficiency of wheel.
69	1.000	1.024	17.23	4	112.50	34.45	46.84	69.59
67	1.000	1.009	17.50	3	152.67	34.22	55.80	82.16
68	1.000	1.008	17.57	2	163.00	34.25	57.31	83.98
66	1.000	1.005	17.46	3	164.00	34.05	56.90	84.40
65	1.000	0.992	17.47	3	172.67	33.60	55.92	84.00
64	1.000	0.979	17.48	3	181.33	33.17	54.53	82.93
63	1.000	0.965	17.46	4	189.25	32.67	52.53	81.21
62	1.000	0.946	17.47	4	213.00	32.06	49.27	77.57
61	1.000	0.688	17.59	4	251.50	23.39	34.91	74.81
60	1.000	0.631	17.86	4	298.00	21.62	.....	.....
59	0.889	0.933	17.50	3	113.67	31.63	45.23	72.04
58	0.889	0.934	17.52	3	135.67	31.70	50.84	80.72
57	0.889	0.931	17.54	3	146.33	31.60	52.80	84.01
56	0.889	0.925	17.53	3	153.67	31.40	53.32	85.41
55	0.889	0.922	17.44	4	156.50	31.21	52.85	85.62
54	0.889	0.916	17.45	3	160.33	31.00	52.66	85.84
53	0.889	0.909	17.46	3	164.00	30.79	52.35	85.87
52	0.889	0.903	17.47	4	168.00	30.59	52.07	85.92
51	0.889	0.898	17.48	4	172.75	30.42	51.95	86.14
50	0.889	0.893	17.48	4	176.75	30.25	51.52	85.91
49	0.889	0.886	17.49	4	184.00	30.05	51.08	85.69
48	0.889	0.864	17.53	4	206.50	29.31	47.77	81.98
47	0.889	0.795	17.59	4	244.75	27.02	33.97	63.02
46	0.889	0.572	17.91	4	298.75	19.61	.....	.....
45	0.741	0.809	17.57	4	106.00	27.48	39.23	71.65
44	0.741	0.801	17.57	4	139.50	27.20	45.18	83.35
43	0.741	0.791	17.55	3	152.00	26.85	45.71	85.53
41	0.741	0.784	17.57	4	159.75	26.63	45.82	86.35
40	0.741	0.780	17.60	4	166.25	26.52	46.15	87.18
42	0.741	0.777	17.56	4	171.50	26.40	46.02	87.53
39	0.741	0.770	17.64	4	180.75	26.22	45.99	87.68
38	0.741	0.758	17.67	4	191.50	25.82	44.30	85.61
37	0.741	0.727	17.73	3	210.67	24.81	38.99	78.15
36	0.741	0.698	17.79	3	231.33	23.68	32.11	67.20
35	0.741	0.503	18.03	2	298.00	17.31	.....	.....
34	0.593	0.669	17.84	3	101.00	22.89	32.71	70.63
33	0.593	0.660	17.84	3	122.67	22.61	36.32	79.40
31	0.593	0.644	17.86	3	145.00	22.07	37.57	84.04
30	0.593	0.642	17.85	3	151.33	21.98	37.81	84.97
32	0.593	0.637	17.88	4	158.00	21.83	38.01	85.87
29	0.593	0.632	17.91	4	163.75	21.68	37.88	86.02
28	0.593	0.622	17.91	3	173.00	21.33	36.82	84.98
27	0.593	0.606	17.99	3	188.00	20.82	34.79	81.90
26	0.593	0.537	18.16	4	239.00	18.54	22.11	57.92
25	0.593	0.406	18.05	3	281.00	13.98	.....	.....
24	0.444	0.505	17.91	3	94.00	17.32	23.92	67.99
22	0.444	0.489	17.92	2	130.00	16.78	27.06	79.36
21	0.444	0.486	17.94	3	136.00	16.70	27.68	81.48
23	0.444	0.486	17.92	4	139.25	16.66	27.70	81.82
20	0.444	0.481	17.94	3	145.33	16.50	27.57	82.12
19	0.444	0.472	17.94	4	153.00	16.21	26.90	81.55
18	0.444	0.465	17.95	5	161.40	15.98	26.13	80.34
17	0.444	0.450	17.99	4	178.00	15.46	24.71	78.32
16	0.444	0.402	18.04	4	231.50	13.85	16.07	56.70

TABLE 1.—(*Continued.*)

Number of experiment.	Proportional Part of: Percentage of full opening of speed-gate.	Proportional Part of: Percentage of full discharge of wheel.	Head acting on wheel, in feet.	Duration of experiment, in minutes.	Revolutions of wheel per minute.	Quantity of water discharged by wheel, in cubic feet per second.	Horse-power developed by wheel.	Percentage of efficiency of wheel.
15	0.444	0.325	18.17	4	281.25	11.22	.....	.....
14	0.296	0.325	18.19	3	89.67	11.22	15.35	66.32
13	0.296	0.320	18.21	3	107.00	11.06	16.58	72.60
11	0.296	0.317	18.23	3	114.67	10.97	16.98	74.85
12	0.296	0.316	18.22	3	118.33	10.94	16.97	75.07
10	0.296	0.313	18.24	3	122.67	10.82	17.03	76.07
9	0.296	0.308	18.25	4	130.75	10.68	16.94	76.62
8	0.296	0.304	18.26	4	137.75	10.54	16.57	75.91
7	0.296	0.300	18.30	4	146.50	10.41	16.27	75.29
6	0.296	0.295	18.31	4	154.75	10.23	15.75	74.15
5	0.296	0.291	18.32	4	164.00	10.10	15.17	72.31
4	0.296	0.283	18.33	5	181.20	9.82	14.25	69.81
3	0.296	0.276	18.36	4	191.25	9.58	13.27	66.53
2	0.296	0.266	18.33	4	208.00	9.23	11.55	60.18
1	0.296	0.225	18.37	4	261.00	7.83	.....	.....

NOTE.—For Experiments Nos. 1, 15, 25, 35, 46 and 60, the jacket was loose.

During the above experiments, the weight of the dynamometer, and of that portion of the shaft which was above the lowest coupling was 1 300 lb.

With the flume empty, a strain of 1.0 lb., applied at a distance of 2.4 ft. from the center of the shaft, sufficed to start the wheel.

Pitot tubes for measuring the velocity at the top and bottom of the guides and in the draft-tube. The latter consisted of a movable tube, with velocity and pressure points, introduced through a gland in the side of the draft-tube about 2 ft. below the bottom of the runner. This tube was arranged to move horizontally across the diameter of the draft-tube and also to revolve about its own axis. By means of these two movements, the velocity at any distance from the center of the shaft could be determined, and also the angle of whirl. The latter would necessarily be the angle of the velocity point which gave the highest reading. Both movements of the tube were controlled from the floor above, and, by means of indicators, the exact position of the point was at all times known.

Fig. 2, Plate LVI, is a photograph of the operating floor, showing the brake for measuring the power of the wheel. At the left is the gauge-board and auxiliary apparatus connected to the Pitot tubes below. The water columns were drawn up into view by means of a vacuum. The arrangement of this apparatus is shown diagrammatically in Fig. 3. The position of the columns in the glass tubes was controlled

TABLE 2.—Tests of a 28-inch R. H. Wellman-Seaver-Morgan Company Turbine Wheel, No. 1796.

Date, February 25th, 1909.

Wheel supported by ball-bearing step. Swing-gate. Conical draft-tube.

Number of experiment.	Proportional Part of:		Head acting on wheel, in feet.	Duration of experiment, in minutes.	Revolutions of wheel per minute.	Quantity of water discharged by wheel, in cubic feet per second.	Horse-power developed by wheel.	Percentage of efficiency of wheel.
	Percentage of full opening of speed-gate.	Percentage of full discharge of wheel.						
95	1.077	0.971	17.11	4	153.00	97.00	125.20	66.52
94	1.077	1.017	16.97	3	199.67	101.16	147.66	75.84
93	1.077	1.036	16.94	3	224.33	102.98	156.38	79.04
92	1.077	1.053	16.89	3	239.33	104.50	159.58	79.72
91	1.077	1.061	16.87	3	247.33	105.22	161.17	80.06
89	1.077	1.068	16.81	3	253.67	105.70	161.45	80.12
90	1.077	1.072	16.80	3	259.00	106.08	161.71	80.01
88	1.077	1.079	16.82	4	267.50	106.82	162.15	79.58
87	1.077	1.026	17.05	3	294.67	102.27	125.03	63.23
86	1.000	0.913	17.28	3	147.00	91.65	120.29	66.98
85	1.000	0.957	17.16	2	190.50	95.70	144.34	77.50
84	1.000	0.972	17.14	3	211.67	97.10	152.69	80.89
83	1.000	0.981	17.13	3	225.00	98.03	156.85	82.36
82	1.000	0.990	17.11	3	232.67	98.90	157.96	82.31
80	1.000	0.996	17.09	4	240.25	99.43	160.19	83.13
81	1.000	1.003	17.07	3	247.33	100.07	161.17	83.19
79	1.000	1.004	17.07	4	252.25	100.14	160.55	82.82
78	1.000	1.001	17.13	4	259.00	100.00	157.00	80.82
77	1.000	0.983	17.22	3	268.33	98.43	146.39	76.15
76	1.000	0.911	17.47	4	293.50	91.96	106.75	58.59
106	0.923	0.868	17.32	4	143.25	87.24	115.49	67.39
105	0.923	0.899	17.24	5	176.00	90.12	135.49	76.90
104	0.923	0.920	17.15	4	201.00	91.96	146.21	81.74
101	0.923	0.931	16.98	5	213.20	92.52	148.62	83.66
100	0.923	0.936	16.98	6	220.67	92.96	150.48	84.31
99	0.923	0.942	16.91	4	227.25	93.51	151.52	84.50
102	0.923	0.945	16.98	4	232.00	93.82	151.88	84.31
103	0.923	0.945	17.04	4	235.50	94.14	152.74	83.96
98	0.923	0.945	16.98	4	237.75	93.82	151.32	84.00
97	0.923	0.924	17.02	4	254.50	92.04	138.84	78.15
96	0.923	0.823	17.25	4	288.25	82.47	87.36	54.15
42	0.923	0.870	17.17	3	146.33	87.02	116.20	68.57
41	0.923	0.895	17.10	4	170.00	89.37	130.87	75.51
40	0.923	0.921	17.04	4	202.75	91.80	146.25	82.44
39	0.923	0.928	16.99	3	208.67	92.35	147.99	83.17
35	0.923	0.932	17.03	4	216.25	92.81	150.75	84.10
38	0.923	0.937	16.97	4	220.00	93.20	151.36	84.38
36	0.923	0.939	17.01	4	223.75	93.44	152.58	84.65
34	0.923	0.940	17.02	4	226.25	93.66	150.86	83.45
37	0.923	0.944	16.97	3	238.33	93.90	151.69	83.94
33	0.923	0.921	17.13	4	256.25	92.05	139.80	78.18
32	0.923	0.823	17.27	3	288.00	82.54	87.29	53.99
31	0.923	0.730	17.50	4	334.75	73.75	.....	.....
74	0.846	0.824	17.46	3	158.67	83.15	120.23	73.02
75	0.846	0.836	17.46	3	175.67	84.35	129.91	77.78
72	0.846	0.861	17.34	5	202.20	86.50	143.40	84.30
70	0.846	0.865	17.33	4	209.00	86.95	145.69	85.25
71	0.846	0.868	17.33	3	215.00	87.24	147.27	85.89
73	0.846	0.870	17.34	4	219.25	87.47	148.19	86.15
69	0.846	0.869	17.32	4	221.25	87.32	147.53	86.01
68	0.846	0.866	17.33	4	227.75	87.02	144.96	84.76
67	0.846	0.858	17.36	4	231.75	86.25	140.48	82.73
66	0.846	0.845	17.39	4	243.75	85.11	132.98	79.22
65	0.846	0.828	17.44	4	256.50	83.44	124.39	75.37

TABLE 2.—(*Continued.*)

Number of experiment.	Proportional Part of:		Head acting on wheel, in feet.	Duration of experiment, in minutes.	Revolutions of wheel per minute.	Quantity of water discharged by wheel, in cubic feet per second.	Horse-power developed by wheel.	Percentage of efficiency of wheel.
	Percentage of full opening of speed-gate.	Percentage of full discharge of wheel.						
64	0.846	0.754	17.59	3	282.00	76.31	85.47	56.15
30	0.769	0.750	17.38	3	141.00	75.52	102.56	68.90
26	0.769	0.766	17.35	3	166.00	77.02	115.72	76.36
27	0.769	0.779	17.32	3	183.00	78.24	124.24	80.84
25	0.769	0.789	17.31	3	194.00	79.25	129.36	83.15
29	0.769	0.793	17.25	4	200.75	79.48	131.42	84.52
28	0.769	0.792	17.26	4	206.00	79.40	131.11	84.36
24	0.769	0.773	17.38	4	226.25	77.82	123.43	80.47
23	0.769	0.735	17.49	3	251.33	74.17	106.64	72.49
22	0.769	0.690	17.56	3	269.67	69.82	81.73	58.78
21	0.769	0.623	17.68	4	323.75	63.27	.....	.....
20	0.615	0.617	17.91	4	139.50	63.07	88.79	69.31
16	0.615	0.627	17.79	3	158.33	63.81	95.97	74.55
17	0.615	0.634	17.80	3	171.33	64.55	101.26	77.71
15	0.615	0.638	17.73	4	179.50	64.82	103.37	79.31
18	0.615	0.636	17.77	4	183.00	64.74	103.16	79.07
19	0.615	0.634	17.80	2	188.00	64.61	102.56	78.64
14	0.615	0.627	17.72	4	194.50	63.74	100.21	78.24
13	0.615	0.596	17.77	3	218.67	60.60	92.78	75.97
12	0.615	0.563	17.79	4	243.00	57.29	73.65	63.72
11	0.615	0.519	17.92	4	312.00	53.00	.....	.....
10	0.462	0.452	17.34	3	117.33	45.40	56.90	63.73
5	0.462	0.453	17.02	3	136.00	45.65	61.83	70.17
7	0.462	0.461	17.08	4	146.75	46.04	64.94	72.82
6	0.462	0.462	17.05	4	152.00	46.04	66.34	74.52
4	0.462	0.462	16.92	4	155.25	45.90	65.88	74.79
9	0.462	0.459	17.16	3	162.00	45.94	66.78	74.69
8	0.462	0.457	17.15	3	166.67	45.69	65.67	73.90
3	0.462	0.451	16.98	4	172.25	44.83	62.65	72.57
2	0.462	0.432	17.05	4	217.50	43.04	52.74	63.37
1	0.462	0.404	17.18	5	282.40	40.40	.....	.....

NOTE.—During the above experiments, the weight of the dynamometer, and of that portion of the shaft which was above the lowest coupling, was 2 600 lb.

With the flume empty, a strain of 0.5 lb., applied at a distance of 3.2 ft. from the center of the shaft, sufficed to start the wheel.

by three-way cocks. By connecting a set of tubes with the vacuum tank, the columns were drawn above the floor level, after which, by closing the cock, they were held in a stationary position. If drawn too high they could be dropped back by opening the cock to the atmosphere. After the columns were once adjusted and the cocks closed, they would stand indefinitely without attention.

This apparatus for the measurement of velocities has been described at some length for the benefit of those who may be interested in the same line of experiments. Lack of space precludes the publication of any of the data obtained, although they are of great interest to the designer, and throw much light on some very obscure points in the theory of turbine design.

TABLE 3.—Tests of a 30-inch R. H. Wellman-Seaver-Morgan Company Turbine Wheel, No. 1797.

Date, February 26th and 27th, 1909.

Wheel supported by ball-bearing step. Swing-gate. Conical draft-tube.

Number of experiment.	Proportional Part of: Percentage of full opening of speed-gate.	Proportional Part of: Percentage of full discharge of wheel.	Head acting on wheel, in feet.	Duration of experiment, in minutes.	Revolutions of wheel per minute.	Quantity of water discharged by wheel, in cubic feet per second.	Horse-power developed by wheel.	Percentage of efficiency of wheel.
82	1.000	0.942	17.06	3	149.00	94.89	126.41	68.85
81	1.000	0.950	16.99	3	160.33	95.52	131.16	71.26
80	1.000	0.963	17.00	4	190.50	96.85	144.30	77.28
75	1.000	0.977	17.02	4	210.50	98.34	150.52	79.30
74	1.000	0.984	16.95	4	220.00	98.81	153.31	80.72
76	1.000	0.992	17.01	4	227.25	99.75	155.61	80.87
78	1.000	0.994	16.94	4	229.50	99.75	155.76	81.28
77	1.000	0.997	16.98	4	231.25	100.14	155.55	80.66
79	1.000	1.002	16.90	3	239.67	100.45	156.85	81.47
73	1.000	1.003	16.94	4	254.75	100.70	154.37	79.80
72	1.000	0.871	17.13	3	287.00	87.93	86.96	50.91
71	1.000	0.728	17.44	3	317.67	74.17	.....	.....
52	0.923	0.875	17.08	3	131.33	88.14	111.42	65.26
51	0.923	0.896	16.95	3	156.33	89.97	127.89	73.95
50	0.923	0.913	16.93	4	188.25	91.66	142.59	81.02
44	0.923	0.920	16.89	4	201.25	92.20	146.34	82.86
45	0.923	0.927	16.85	4	211.50	92.81	148.67	83.83
46	0.923	0.931	16.86	4	217.00	93.20	149.91	84.12
49	0.923	0.934	16.83	4	220.50	93.42	150.99	84.68
47	0.923	0.936	16.83	4	223.75	93.60	151.86	85.00
48	0.923	0.937	16.83	3	226.33	93.74	150.87	84.32
43	0.923	0.929	16.88	4	241.75	93.13	146.49	82.17
42	0.923	0.788	17.21	4	276.25	79.76	83.70	53.77
41	0.923	0.670	17.44	4	312.50	68.26	.....	.....
70	0.846	0.868	17.07	4	192.75	87.47	142.50	84.15
67	0.846	0.875	17.10	4	205.00	88.22	147.83	86.41
68	0.846	0.876	17.06	4	209.50	88.22	148.53	87.02
66	0.846	0.876	17.11	4	212.00	88.37	149.02	86.91
69	0.846	0.877	17.05	3	213.00	88.30	148.43	86.94
65	0.846	0.876	17.13	5	213.80	88.37	147.70	86.03
64	0.846	0.875	17.13	5	215.80	88.30	146.46	85.38
63	0.808	0.810	17.33	4	143.50	82.24	114.78	71.01
62	0.808	0.834	17.25	3	177.00	84.44	135.14	81.81
61	0.808	0.843	17.24	5	194.80	85.35	142.83	85.59
60	0.808	0.847	17.21	3	201.00	85.73	144.94	86.62
59	0.808	0.848	17.19	3	206.00	85.73	146.05	87.39
58	0.808	0.847	17.19	4	208.25	85.65	145.12	86.91
57	0.808	0.845	17.19	3	210.00	85.41	148.80	86.36
56	0.808	0.840	17.22	3	216.67	85.05	141.80	85.37
55	0.808	0.825	17.25	4	224.75	83.53	136.19	83.34
54	0.808	0.807	17.29	4	236.50	81.80	128.98	80.41
53	0.808	0.695	17.51	4	264.25	70.97	80.06	56.81
40	0.769	0.755	17.29	4	118.50	76.53	93.35	62.21
39	0.769	0.776	17.21	4	146.50	78.47	110.97	72.46
35	0.769	0.800	17.03	4	174.75	80.49	127.07	81.74
36	0.769	0.806	17.03	4	190.75	81.13	135.24	86.31
37	0.769	0.805	17.10	4	197.25	81.21	137.46	87.28
38	0.769	0.805	17.15	4	200.50	81.28	137.29	86.84
34	0.769	0.803	16.83	4	199.50	80.34	132.98	86.72
33	0.769	0.788	16.93	4	211.75	79.04	128.32	84.55
32	0.769	0.745	16.93	4	232.50	74.72	112.71	78.56
31	0.769	0.660	17.36	4	257.50	67.02	78.02	59.13
30	0.769	0.579	17.60	3	306.00	59.27	.....	.....
29	0.615	0.610	17.69	3	100.67	62.60	69.54	55.37

TABLE 3.—(*Continued.*)

Number of experiment.	Proportional Part of: Percentage of full opening of speed-gate.	Proportional Part of: Percentage of full discharge of wheel.	Head acting on wheel, in feet.	Duration of experiment, in minutes.	Revolutions of wheel, per minute.	Quantity of water discharged by wheel, in cubic feet per second.	Horse-power developed by wheel.	Percentage of efficiency of wheel.
28	0.615	0.621	17.64	3	127.67	63.60	85.10	66.89
25	0.615	0.643	17.60	3	167.33	65.76	104.44	79.57
27	0.615	0.644	17.57	4	175.50	65.83	107.41	81.89
24	0.615	0.643	17.61	4	178.00	65.76	107.86	82.13
26	0.615	0.641	17.59	3	179.67	65.55	106.70	81.60
23	0.615	0.482	17.96	3	296.67	49.84	.....	.....
20	0.615	0.634	17.88	4	155.00	65.37	101.44	76.53
21	0.615	0.634	17.80	5	151.40	65.22	98.17	74.56
22	0.615	0.639	17.69	3	160.33	65.50	102.98	78.37
19	0.615	0.644	18.08	5	174.20	66.80	111.89	81.69
18	0.615	0.643	18.11	3	180.00	66.73	112.35	81.97
17	0.615	0.638	18.07	4	183.50	66.19	111.20	81.98
16	0.615	0.636	18.03	4	185.00	65.90	109.86	81.53
15	0.615	0.630	17.96	3	190.00	65.15	109.38	82.43
14	0.615	0.609	17.81	4	201.00	62.66	103.53	81.80
13	0.615	0.574	17.79	4	216.75	59.07	91.94	77.15
12	0.615	0.553	17.54	5	245.20	56.46	74.29	66.15
11	0.615	0.483	17.71	5	294.60	49.58	.....	.....
8	0.462	0.475	17.82	3	104.33	48.95	58.16	58.80
7	0.462	0.483	17.75	4	139.75	49.58	71.98	72.12
6	0.462	0.482	17.76	3	147.00	49.51	73.93	74.14
5	0.462	0.481	17.80	4	156.75	49.45	75.99	76.12
9	0.462	0.477	17.93	2	162.00	49.22	76.57	76.51
10	0.462	0.472	17.97	3	167.00	48.78	75.90	76.35
4	0.462	0.464	17.88	5	173.60	47.80	73.64	75.97
3	0.462	0.436	18.05	4	216.50	45.13	65.60	71.01
2	0.462	0.415	18.08	4	246.25	43.02	44.77	50.75
1	0.462	0.381	18.27	4	280.50	39.75	.....	.....

NOTE.—The jacket was loose for Experiments Nos. 1, 11, 23, 30, 41 and 71.

During the above experiments, the weight of the dynamometer, and of that portion of the shaft which was above the lowest coupling was 2 600 lb.

With the flume empty, a strain of 0.5 lb., applied at a distance of 3.2 ft. from the center of the shaft, sufficed to start the wheel.

Figs. 4 to 19, inclusive, are performance curves plotted from the test reports, on the common basis of 1 ft. head. The head at the testing flume fluctuates, and it is therefore necessary to reduce all readings to a common basis. With constant efficiency, the brake-horse-power varies as $\sqrt{h^3}$, and the speed and discharge as $\sqrt{h}$. This principle is true, theoretically, and has been repeatedly proven, experimentally, and all the computations have been made upon this basis.

Three sets of curves have been plotted for each of the five wheels. The first in order for each wheel is a set of brake-horse-power and efficiency curves for all gate openings plotted to speed. Each curve is designated by the gate opening which it represents. In addition to power and speed, the writer usually plots the discharge curves, but

TABLE 4.—Tests of a 31-inch R. H. Wellman-Seaver-Morgan Company Turbine Wheel, No. 1799.

Date, March 2d and 3d, 1909.

Wheel supported by ball-bearing step. Swing-gate. Conical draft-tube.

Number of experiment.	Proportional Part of: Percentage of full opening of speed-gate.	Proportional Part of: Percentage of full discharge of wheel.	Head acting on wheel, in feet.	Duration of experiment, in minutes.	Revolutions of wheel per minute.	Quantity of water discharged by wheel, in cubic feet per second.	Horse-power developed by wheel.	Percentage of efficiency of wheel.
49	1.000	1.049	17.15	3	134.67	79.75	115.81	74.66
48	1.000	1.038	17.15	3	147.00	78.95	119.29	77.68
47	1.000	1.025	17.19	3	165.67	78.02	123.40	81.13
46	1.000	1.024	17.18	2	174.00	77.91	124.34	81.91
45	1.000	1.018	17.19	3	178.33	77.52	124.19	82.18
44	1.000	1.013	17.19	3	183.33	77.16	124.35	82.66
43	1.000	0.012	17.19	4	186.25	77.09	124.07	82.56
42	1.000	1.009	17.18	4	189.50	76.82	123.94	82.81
41	1.000	1.007	17.19	3	193.00	76.67	123.89	82.89
40	1.000	1.006	17.12	4	195.50	76.44	123.13	82.96
39	1.000	1.002	17.09	4	200.25	76.10	122.48	83.04
38	1.000	0.999	17.10	4	206.25	75.87	122.41	83.19
37	1.000	0.997	17.07	3	210.33	75.66	121.01	82.62
36	1.000	0.993	17.09	4	219.00	75.38	119.36	81.70
35	1.000	0.906	17.24	4	258.75	69.11	78.35	57.98
34	1.000	0.744	17.45	4	302.25	57.07	.....	.....
32	0.883	0.909	17.18	3	129.33	69.23	101.03	74.90
31	0.883	0.904	17.20	4	154.25	68.82	111.16	82.81
33	0.883	0.901	17.19	4	165.00	68.60	113.91	85.18
30	0.883	0.896	17.20	3	172.33	68.25	114.80	86.23
29	0.883	0.892	17.21	4	181.50	67.97	116.51	87.82
27	0.883	0.888	17.25	3	188.00	67.77	117.27	88.45
26	0.883	0.886	17.30	4	194.00	67.71	117.49	88.44
28	0.883	0.885	17.23	4	197.25	67.43	117.06	88.85
25	0.883	0.883	17.31	4	201.75	67.49	116.07	87.61
24	0.883	0.857	17.28	4	213.25	65.42	109.77	85.62
23	0.883	0.818	17.31	4	227.00	62.53	96.23	78.39
22	0.883	0.761	17.42	4	244.25	58.32	73.96	64.19
21	0.883	0.620	17.65	4	291.75	47.83	.....	.....
69	0.750	0.835	17.28	4	124.75	63.73	91.41	73.19
68	0.750	0.839	17.23	4	148.75	63.93	103.59	82.93
67	0.750	0.834	17.23	4	168.75	63.60	109.35	87.99
64	0.750	0.830	17.24	4	179.25	63.26	110.72	89.52
65	0.750	0.829	17.23	4	182.00	63.20	110.77	89.69
63	0.750	0.828	17.25	4	186.25	63.12	111.66	90.43
66	0.750	0.824	17.23	3	188.67	62.85	110.83	90.24
62	0.750	0.821	17.29	4	191.75	62.72	110.32	89.70
61	0.750	0.809	17.30	4	197.50	61.81	107.64	88.76
60	0.750	0.796	17.34	4	203.25	60.86	104.62	87.42
59	0.750	0.768	17.38	4	212.50	58.80	96.52	83.28
58	0.750	0.693	17.55	4	237.00	53.31	71.76	67.64
57	0.750	0.573	17.75	4	287.00	44.32	.....	.....
20	0.667	0.726	17.27	4	106.25	55.43	70.78	65.20
19	0.667	0.737	17.26	3	148.67	56.25	90.93	82.59
18	0.667	0.739	17.25	4	161.00	56.39	95.55	86.62
17	0.667	0.738	17.24	4	168.50	56.31	96.94	88.05
16	0.667	0.733	17.27	3	173.67	55.92	96.76	88.35
15	0.667	0.722	17.30	4	179.25	55.17	95.52	88.25
14	0.667	0.699	17.34	4	189.50	53.48	91.81	87.30
13	0.667	0.671	17.39	4	201.75	51.37	85.52	84.42
11	0.667	0.508	17.63	4	280.75	39.17	.....	.....
8	0.500	0.554	17.82	3	117.00	42.95	60.23	69.39
7	0.500	0.546	17.91	3	135.00	42.46	65.40	75.84
10	0.500	0.548	17.67	3	151.00	42.28	68.58	80.95

TABLE 4.—(*Continued.*)

Number of experiment.	Proportional Part of:		Head acting on wheel, in feet.	Duration of experiments, in minutes.	Revolutions of wheel per minute.	Quantity of water discharged by wheel, in cubic feet per second.	Horse-power developed by wheel.	Percentage of efficiency of wheel.
	Percentage of full opening of speed-gate.	Percentage of full discharge of wheel.						
6	0.500	0.548	18.05	4	157.50	42.72	71.54	81.80
9	0.500	0.547	17.71	4	157.00	42.28	69.41	81.73
5	0.500	0.539	18.12	5	167.60	42.11	71.05	82.10
4	0.500	0.512	18.13	4	187.00	40.02	67.95	82.58
3	0.500	0.488	18.18	4	213.00	38.21	68.05	73.68
2	0.500	0.460	18.07	4	232.00	35.92	42.15	57.26
1	0.500	0.402	18.20	3	275.00	31.50	.....	.....
52	0.333	0.362	18.18	3	96.00	28.35	34.88	59.68
55	0.333	0.361	18.01	3	117.33	28.10	39.08	68.09
51	0.333	0.348	18.19	4	133.00	27.25	40.27	71.64
54	0.333	0.347	18.21	3	139.67	27.20	40.60	72.28
53	0.333	0.340	18.22	3	143.67	26.62	39.15	71.18
56	0.333	0.333	18.06	4	148.00	26.00	37.64	70.69
50	0.333	0.316	18.26	3	201.67	24.82	36.64	71.28

Note.—For Experiments Nos. 1, 11, 21, 34, and 57, the jacket was loose.

During the above experiments, the weight of the dynamometer and of that portion of the shaft which was above the lowest coupling was 2 600 lb.

With the flume empty, a strain of 1.0 lb., applied at a distance of 3.2 ft. from the center of the shaft, sufficed to start the wheel.

they have been omitted from this paper in order to economize space.* When included, such a diagram might be termed in common parlance a "personality chart," because it embodies all the characteristics and peculiarities of the wheel. From it separate performance curves can be plotted for any speed whatsoever within the range of the test. The heavy speed line drawn through each of these charts indicates the "normal speed" of the wheel, that is, the speed at which it gives its maximum efficiency.

The second set of curves shows the performance of the wheels along these heavy lines, or at normal speed. The gate openings are also plotted to discharge, so that it is possible to read the efficiency at any fractional part of the full gate opening as well as at any fractional part of the full discharge of the wheel.

The first set of curves shows, in addition to the information already referred to, a scale of values of ϕ, in the equation,

$$W_1 = \phi \sqrt{2\,gh},$$

where W_1 is the peripheral speed of the wheel, based on its rated

* For full details of this method of plotting test curves, see Chapter XVI of "Water Power Engineering," by Daniel W. Mead, M. Am. Soc. C. E.

TABLE 5.—Tests of a 31-inch R. H. Wellman-Seaver-Morgan Company Turbine Wheel, No. 1800.

Date, March 4th and 5th, 1909.

Wheel supported on ball-bearing step. Swing-gate. Conical draft-tube.

Number of experiment.	Proportional Part of: Percentage of full opening of speed-gate.	Proportional Part of: Percentage of full discharge of wheel.	Head acting on wheel, in feet.	Duration of experiment, in minutes.	Revolutions of wheel per minute.	Quantity of water discharged by wheel, in cubic feet per second.	Horse-power developed by wheel.	Percentage of efficiency of wheel.
65	1.000	1.052	17.38	3	112.33	64.87	82.99	64.91
64	1.000	1.040	17.41	3	133.33	64.20	91.24	71.98
63	1.000	1.025	17.43	4	154.25	63.33	96.21	76.85
62	1.000	1.014	17.42	3	169.67	62.60	98.64	79.76
61	1.000	1.009	17.40	3	183.33	62.25	99.92	81.34
60	1.000	1.006	17.42	4	193.25	62.13	100.65	82.00
59	1.000	1.004	17.43	4	201.00	62.01	101.08	82.42
58	1.000	1.002	17.44	4	207.00	61.88	101.54	82.96
56	1.000	1.000	17.35	4	211.00	61.65	100.95	83.22
57	1.000	0.999	17.39	4	216.75	61.65	101.07	83.13
55	1.000	0.998	17.36	4	221.50	61.53	100.60	83.05
54	1.000	0.995	17.38	4	247.50	61.40	97.42	80.50
53	1.000	0.951	17.45	4	266.75	58.80	80.77	69.41
52	1.000	0.730	17.71	4	321.25	45.44	.....	.....
51	0.883	0.889	17.48	4	132.25	55.02	79.29	72.69
50	0.883	0.888	17.48	4	147.00	54.91	83.68	76.88
49	0.883	0.885	17.47	4	165.00	54.71	88.93	82.04
48	0.883	0.879	17.39	4	179.25	54.24	91.18	85.24
47	0.883	0.876	17.42	5	189.20	54.11	91.66	85.75
44	0.883	0.873	17.54	4	197.50	54.11	93.29	86.67
43	0.883	0.873	17.40	4	204.25	53.85	92.77	87.30
45	0.883	0.872	17.40	3	209.00	53.79	92.40	87.05
46	0.883	0.871	17.42	4	215.50	53.79	92.66	87.19
42	0.883	0.858	17.40	3	225.00	52.92	88.57	84.81
41	0.883	0.810	17.45	3	248.00	50.06	75.09	75.80
40	0.883	0.609	17.68	3	314.00	37.87	.....	.....
75	0.733	0.801	17.76	4	158.50	49.93	81.59	81.13
73	0.733	0.798	17.73	3	183.33	49.70	86.60	86.66
72	0.733	0.797	17.71	3	190.33	49.63	87.60	87.88
71	0.733	0.796	17.72	3	197.00	49.57	88.28	88.62
74	0.733	0.795	17.73	3	199.00	49.50	87.97	88.39
70	0.733	0.793	17.72	3	201.67	49.37	87.93	88.63
69	0.733	0.788	17.70	4	204.50	49.02	86.69	88.10
68	0.733	0.775	17.71	4	213.50	48.27	84.04	86.69
67	0.733	0.722	17.79	3	234.00	45.04	70.85	77.97
66	0.733	0.544	18.03	3	308.67	34.15	.....	.....
38	0.667	0.751	17.58	3	171.00	46.58	78.70	84.75
37	0.667	0.750	17.57	4	184.00	46.52	81.34	87.75
39	0.667	0.749	17.56	4	187.00	46.45	81.54	88.14
35	0.667	0.749	17.59	4	185.50	46.45	80.88	87.29
36	0.667	0.747	17.59	4	188.75	46.38	81.73	88.33
34	0.667	0.742	17.58	4	192.50	46.02	80.44	87.67
33	0.667	0.731	17.59	4	198.25	45.37	78.04	86.22
32	0.667	0.753	17.70	3	127.67	46.87	68.04	72.32
31	0.667	0.752	17.68	3	144.67	46.76	72.72	77.56
29	0.667	0.752	17.66	4	164.25	46.76	77.59	82.85
30	0.667	0.753	17.68	3	178.67	46.82	81.15	86.44
28	0.667	0 750	17.62	3	189.00	46.57	82.41	88.56
27	0.667	0.741	17.62	3	193.67	46.02	80.93	88.00
26	0.667	0.731	17.64	3	201.00	45.44	79.12	87.04
25	0.667	0.725	17.63	3	204.00	45.05	77.83	86.41
24	0.667	0.709	17.64	3	213.33	44.05	74.93	85.03
23	0.667	0.678	17.70	4	224.50	42.23	67.98	80.19
22	0.667	0.506	17.85	3	303.33	31.61	.....	.....

TABLE 5.—(*Continued.*)

Number of experiment.	Proportional Part of: Percentage of full opening of speed-gate.	Proportional Part of: Percentage of full discharge of wheel	Head acting on wheel, in feet.	Duration of experiment, in minutes.	Revolutions of wheel per minute.	Quantity of water discharged by wheel, in cubic feet per second.	Horse-power developed by wheel.	Percentage of efficiency of wheel.
21	0.500	0.566	17.72	2	124.00	35.27	50.31	70.98
20	0.500	0.560	17.74	3	136.33	34.87	52.84	75.32
19	0.500	0.554	17.71	4	146.50	34.50	53.23	76.82
18	0.500	0.543	17.72	4	155.75	33.82	52.82	77.72
17	0.500	0.535	17.74	3	166.00	33.31	52.27	78.00
16	0.500	0.521	17.77	3	176.67	32.47	51.36	78.48
15	0.500	0.507	17.79	3	189.00	31.61	50.36	78.97
14	0.500	0.494	17.83	4	200.25	30.87	48.51	77.71
13	0.500	0.481	17.86	3	214.00	30.08	45.36	74.45
12	0.500	0.464	17.89	4	228.75	29.02	41.56	70.58
11	0.500	0.380	18.00	4	299.50	23.87	.....	.....
8	0.333	0.360	18.09	4	112.00	22.65	30.52	65.68
7	0.333	0.351	18.12	3	127.00	22.10	31.53	69.43
6	0.333	0.350	18.13	3	136.00	22.02	32.12	70.95
5	0.333	0.346	18.22	4	141.50	21.84	31.71	70.26
4	0.333	0.340	18.24	4	148.00	21.47	31.37	70.63
3	0.333	0.332	18.31	4	154.75	21.03	30.93	70.82
2	0.333	0.323	18.25	4	165.25	20.42	30.02	71.04
10	0.333	0.307	18.17	4	212.50	19.33	28.31	71.08
9	0.333	0.300	18.15	3	231.33	18.93	25.22	64.72
1	0.333	0.248	18.35	4	287.25	15.74	.....	.....

NOTE.—The jacket was loose for Experiments Nos. 1, 11, 22, 40, 52, and 66.

During the above experiments, the weight of the dynamometer and of that portion of the shaft which was above the lowest coupling was 2 600 lb.

With the flume empty, a strain or 0.5 lb., applied at a distance of 3.2 ft. from the center of the shaft, sufficed to start the wheel.

diameter. The third set of curves shows the performance of the wheels at speeds corresponding to several different values of ϕ. When a wheel operates at constant speed under a fluctuating head, the value of ϕ is continually changing. A comparison of these curves, therefore, is a very instructive study, because it shows how the efficiency varies as the head changes. The change in efficiency corresponding to a given change in ϕ is very different for different types of runner.

High-Speed Wheels.—In the use of the term "high-speed wheel," there seems to be much confusion. In the sense in which it is used here, the term does not apply necessarily to wheels of high peripheral speed, although a high value of ϕ is usually an attribute of a "high-speed wheel." As the term is used here, the wheel which, power for power, under a constant head, runs at the highest number of revolutions, is the highest-speed wheel. A high-speed wheel is one which will develop any given quantity of power at a high speed, relative to other wheels developing the same power under the same head. It is not

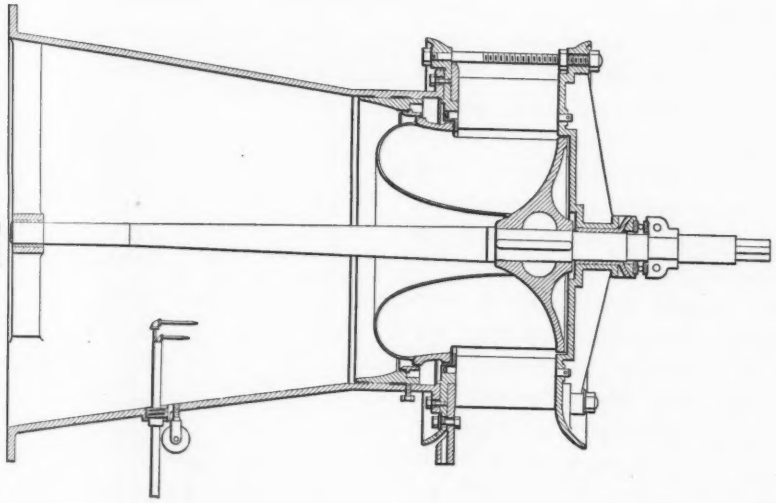


Fig. 1.

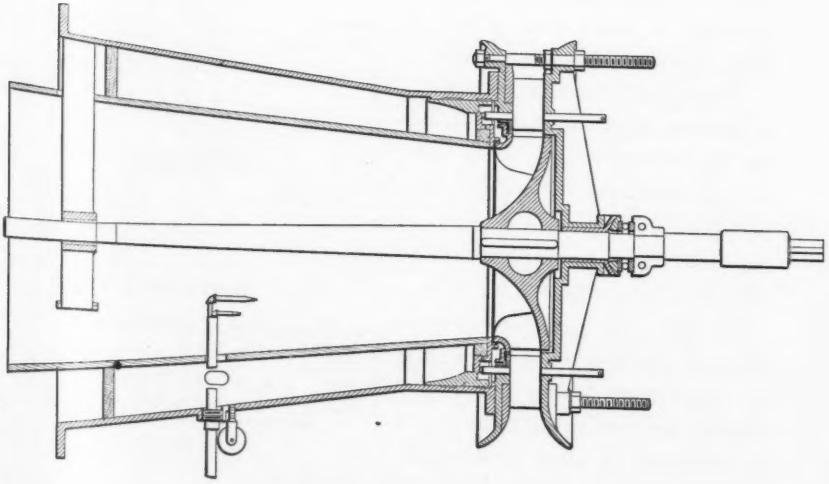


Fig. 2.

possible to compare the speeds of wheels upon the basis of diameter, because the power is not a consistent function of the diameter, except in wheels of homologous design. To determine which of two wheels is the higher-speed, it is usually necessary, unless they differ widely, to reduce them to a common basis of power and head.

Suppose, for example, that two types of runner, A and B, are to be compared, and it is known from tests, or otherwise, that some certain size of A (the diameter of which need not be given) will develop a maximum of 2 080 h.p. at 500 rev. per min. under 100 ft. head, whereas an entirely different size of B will develop a maximum of 4 390 h.p. at 580 rev. per min. under 150 ft. head. It is impossible to determine from an inspection of these data which of these types is the higher-speed.

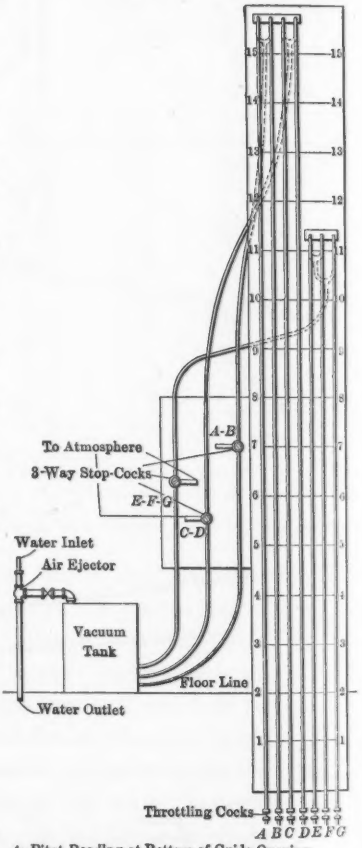
They should first be reduced to a common head, say 1 ft. Dividing the horse-power in each case by $\sqrt{h^3}$ and the speed by $\sqrt{h}$, the result is

A. .2.07 h.p. at 50 rev. per min.

B. .2.37 h.p. at 47.4 rev. per min.

A runs at a higher speed than B, but B develops more power than A. One cannot be sure but that if B were reduced in diameter until it developed the same power as A, it might run at as high a speed as A, or higher.

DIAGRAM OF GAUGE-BOARD AND AUXILIARY VACUUM APPARATUS FOR MEASURING VELOCITIES THROUGH GUIDE OPENINGS AND DRAFT-TUBE



A- Pitot Reading at Bottom of Guide Opening
B- Pressure " " " " " "
C- Pitot " " Top " " "
D- Pressure " " " " " "
E- Pitot " from Movable Point in Draft-Tube
F- Pressure " " " " " "
G- Pressure " " Wall of Draft-Tube at same Elevation as Movable Point.

FIG. 3.

Next reduce them to the same power, say 1 h.p. It is known that the power of wheels of homologous design varies directly as the

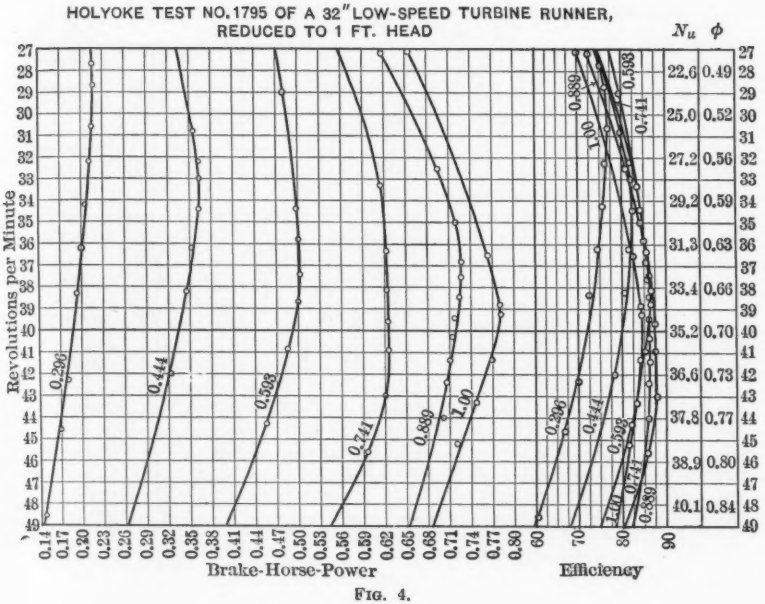

HOLYOKE TEST NO. 1795 OF A 32" LOW-SPEED TURBINE RUNNER,
REDUCED TO 1 FT. HEAD
N_u ϕ
22.6 0.49
25.0 0.52
27.2 0.56
29.2 0.59
31.3 0.63
33.4 0.66
35.2 0.70
36.6 0.73
37.8 0.77
38.9 0.80
40.1 0.84
Revolutions per Minute
Brake-Horse-Power
Efficiency
0.296
0.444
0.593
0.741
0.889
1.00

FIG. 4.

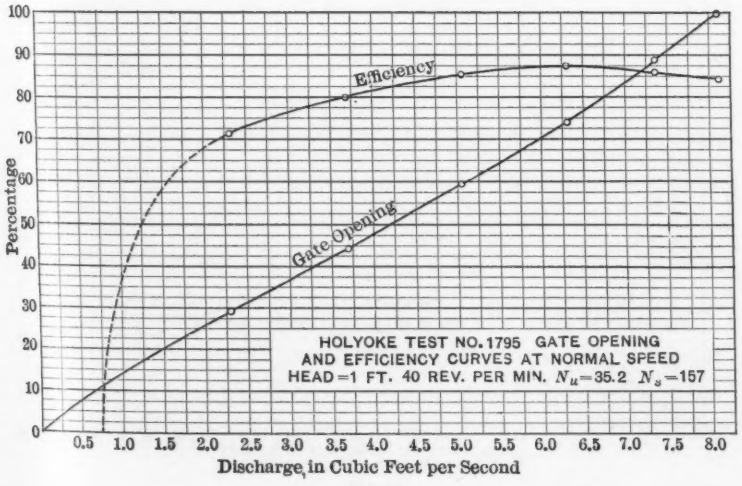

Efficiency
Gate Opening
Percentage
HOLYOKE TEST NO. 1795 GATE OPENING
AND EFFICIENCY CURVES AT NORMAL SPEED
HEAD = 1 FT. 40 REV. PER MIN. N_u = 35.2 N_s = 157
Discharge, in Cubic Feet per Second

FIG. 5.

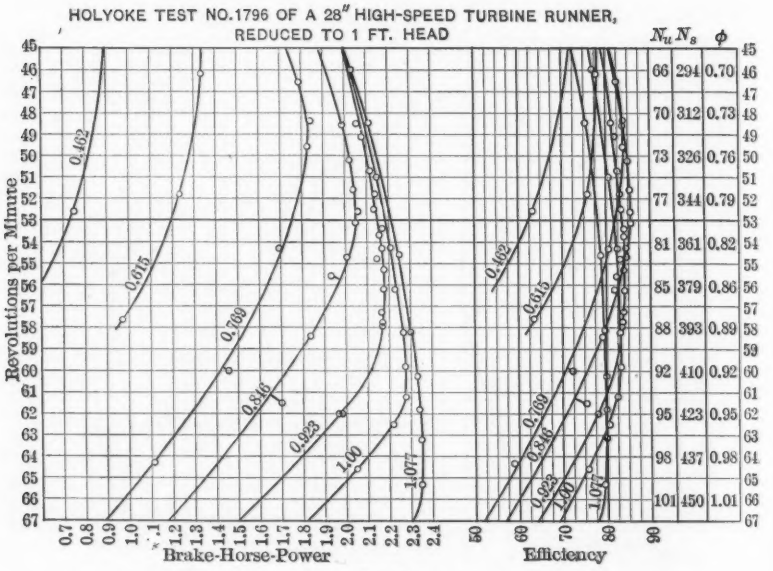

HOLYOKE TEST NO. 1796 OF A 28" HIGH-SPEED TURBINE RUNNER,
REDUCED TO 1 FT. HEAD
N_u N_s ϕ
66 294 0.70
70 312 0.73
73 326 0.76
77 344 0.79
81 361 0.82
85 379 0.86
88 393 0.89
92 410 0.92
95 423 0.95
98 437 0.98
101 450 1.01
Revolutions per Minute
0.462
0.615
0.769
0.846
0.923
1.00
1.077
Brake-Horse-Power
Efficiency

FIG. 6.

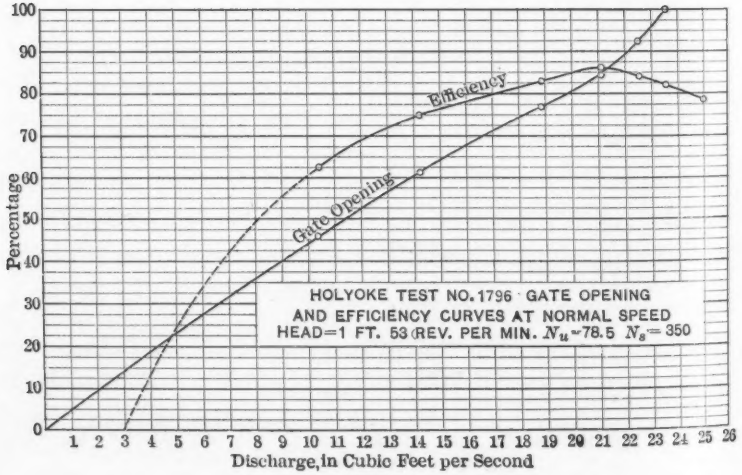

Efficiency
Gate Opening
Percentage
HOLYOKE TEST NO. 1796 · GATE OPENING
AND EFFICIENCY CURVES AT NORMAL SPEED
HEAD=1 FT. 53 REV. PER MIN. N_u=78.5 N_s=350
Discharge, in Cubic Feet per Second

FIG. 7.

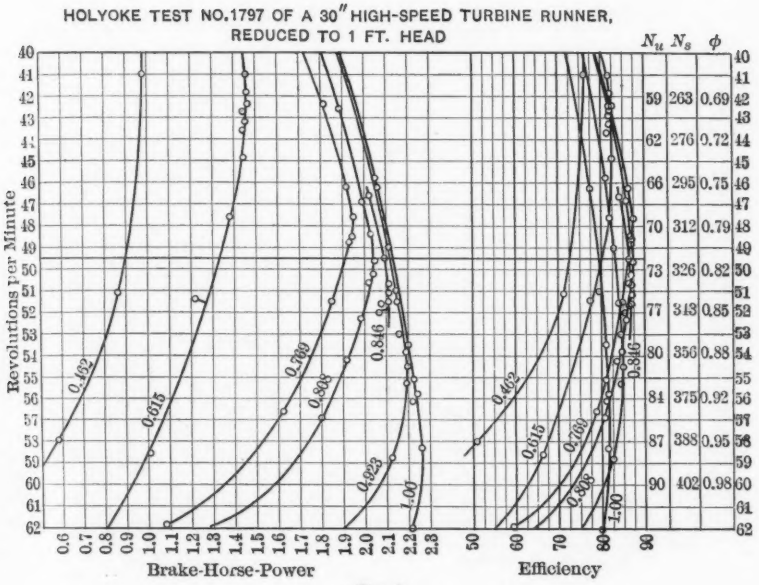

HOLYOKE TEST NO. 1797 OF A 30" HIGH-SPEED TURBINE RUNNER,
REDUCED TO 1 FT. HEAD
N_u N_s ϕ
59 263 0.69
62 276 0.72
66 295 0.75
70 312 0.79
73 326 0.82
77 343 0.85
80 356 0.88
84 375 0.92
87 388 0.95
90 402 0.98
Revolutions per Minute
0.492
0.615
0.769
0.808
0.846
0.923
1.00
Brake-Horse-Power
Efficiency

FIG. 8.

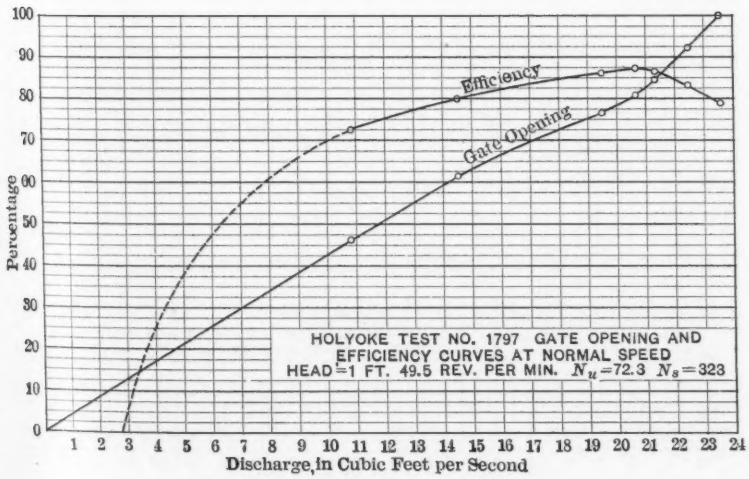

Efficiency
Gate Opening
Percentage
HOLYOKE TEST NO. 1797 GATE OPENING AND
EFFICIENCY CURVES AT NORMAL SPEED
HEAD=1 FT. 49.5 REV. PER MIN. N_u=72.3 N_s=323
Discharge, in Cubic Feet per Second

FIG. 9.

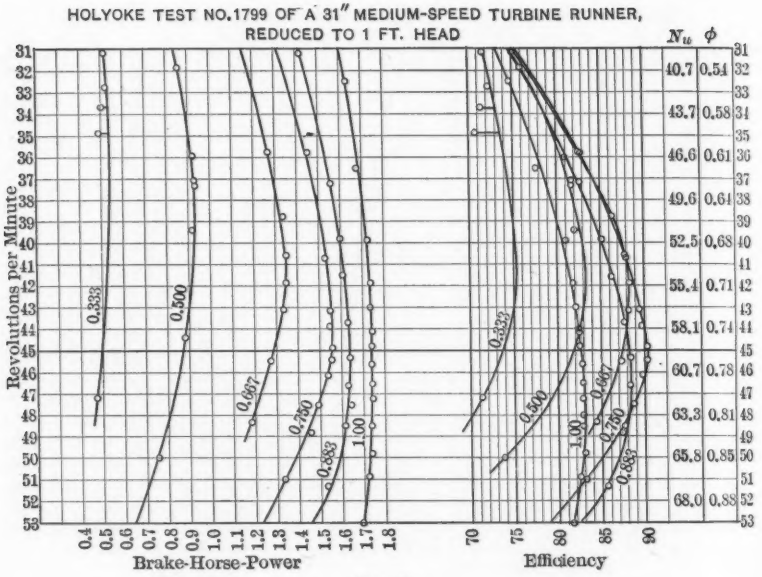

HOLYOKE TEST NO.1799 OF A 31″ MEDIUM-SPEED TURBINE RUNNER,
REDUCED TO 1 FT. HEAD
N_u ϕ
40.7 0.54
43.7 0.58
46.6 0.61
49.6 0.64
52.5 0.68
55.4 0.71
58.1 0.74
60.7 0.78
63.3 0.81
65.8 0.85
68.0 0.88
Revolutions per Minute
Brake-Horse-Power
Efficiency
0.333
0.500
0.667
0.750
0.833
1.00

FIG. 10.

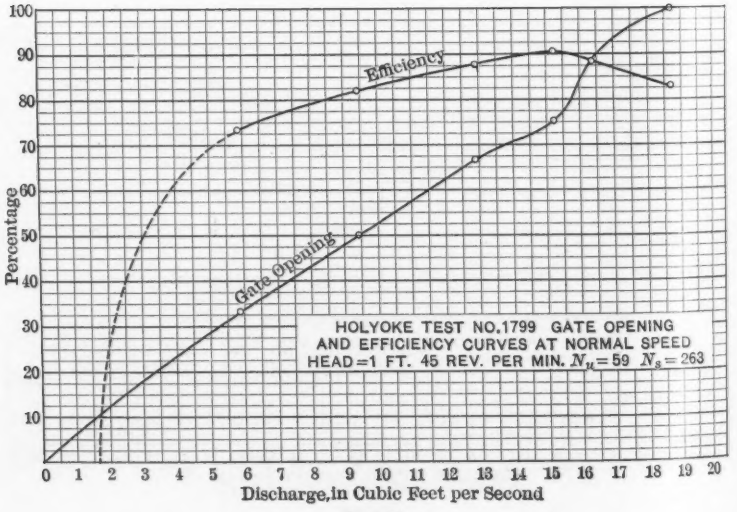

Efficiency
Gate Opening
Percentage
HOLYOKE TEST NO.1799 GATE OPENING
AND EFFICIENCY CURVES AT NORMAL SPEED
HEAD=1 FT. 45 REV. PER MIN. N_H=59 N_S=263
Discharge, in Cubic Feet per Second

FIG. 11.

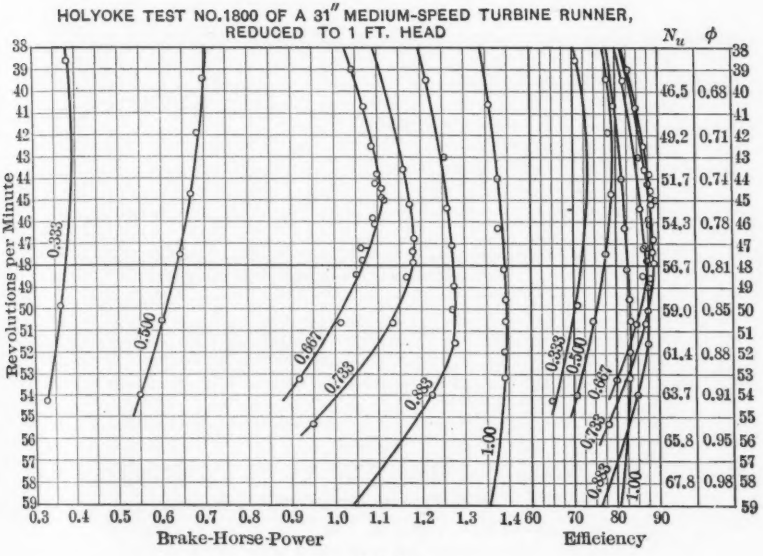

HOLYOKE TEST NO. 1800 OF A 31″ MEDIUM-SPEED TURBINE RUNNER, REDUCED TO 1 FT. HEAD
N_u
ϕ
46.5 0.68
49.2 0.71
51.7 0.74
54.3 0.78
56.7 0.81
59.0 0.85
61.4 0.88
63.7 0.91
65.8 0.95
67.8 0.98
0.333
0.500
0.667
0.733
0.833
1.00
Revolutions per Minute
Brake-Horse-Power
Efficiency

FIG. 12.

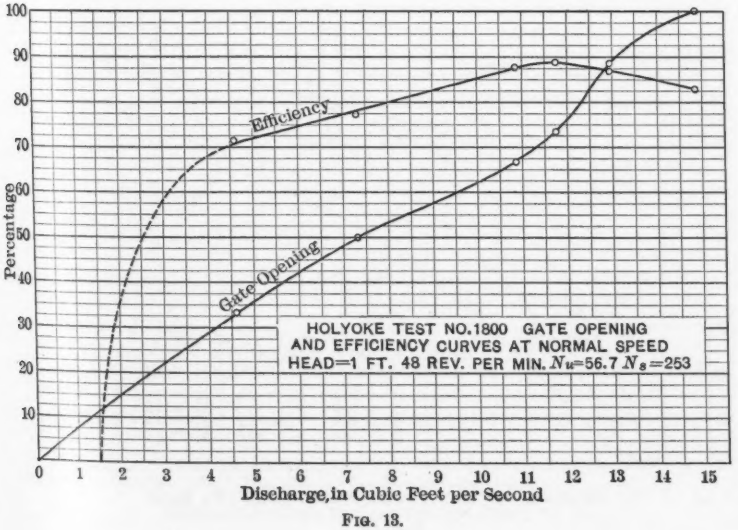

Efficiency
Gate Opening
HOLYOKE TEST NO. 1800 GATE OPENING AND EFFICIENCY CURVES AT NORMAL SPEED HEAD=1 FT. 48 REV. PER MIN. N_u=56.7 N_s=253
Percentage
Discharge, in Cubic Feet per Second

FIG. 13.

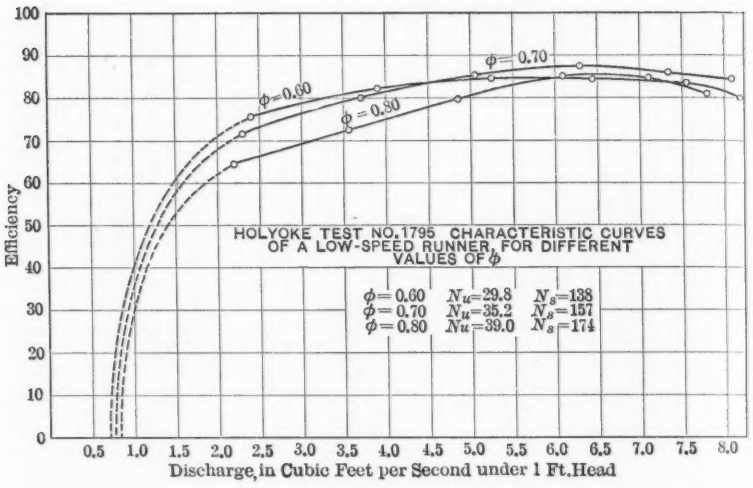

Efficiency
Discharge, in Cubic Feet per Second under 1 Ft. Head
HOLYOKE TEST NO. 1795. CHARACTERISTIC CURVES OF A LOW-SPEED RUNNER, FOR DIFFERENT VALUES OF φ
φ = 0.60 Nu = 29.8 Ns = 138
φ = 0.70 Nu = 35.2 Ns = 157
φ = 0.80 Nu = 39.0 Ns = 174
φ = 0.60
φ = 0.70
φ = 0.80

FIG. 14.

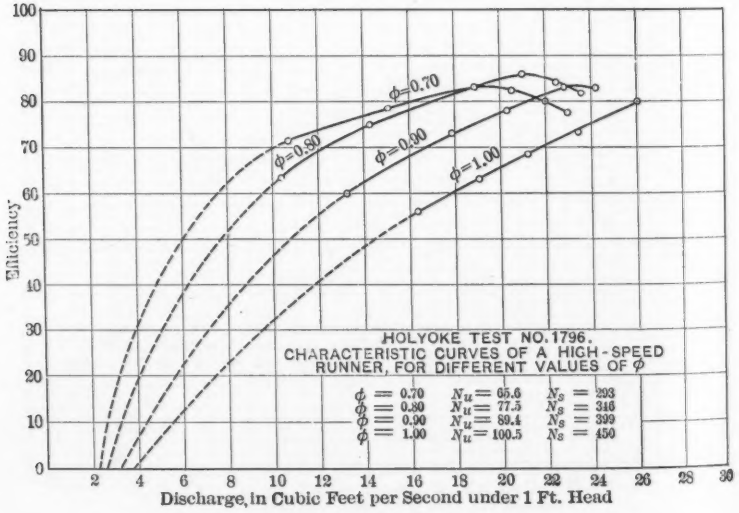

Efficiency
Discharge, in Cubic Feet per Second under 1 Ft. Head
HOLYOKE TEST NO. 1796. CHARACTERISTIC CURVES OF A HIGH-SPEED RUNNER, FOR DIFFERENT VALUES OF φ
φ = 0.70 Nu = 65.6 Ns = 293
φ = 0.80 Nu = 77.5 Ns = 346
φ = 0.90 Nu = 89.4 Ns = 399
φ = 1.00 Nu = 100.5 Ns = 450
φ = 0.70
φ = 0.80
φ = 0.90
φ = 1.00

FIG. 15.

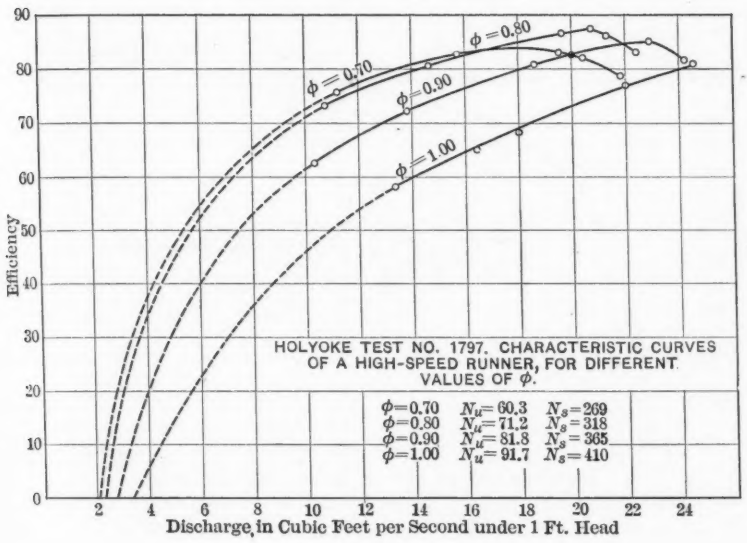

Efficiency
Discharge, in Cubic Feet per Second under 1 Ft. Head
$\phi = 0.70$
$\phi = 0.80$
$\phi = 0.90$
$\phi = 1.00$
HOLYOKE TEST NO. 1797. CHARACTERISTIC CURVES OF A HIGH-SPEED RUNNER, FOR DIFFERENT VALUES OF ϕ.
$\phi = 0.70$ $N_u = 60.3$ $N_s = 269$
$\phi = 0.80$ $N_u = 71.2$ $N_s = 318$
$\phi = 0.90$ $N_u = 81.8$ $N_s = 365$
$\phi = 1.00$ $N_u = 91.7$ $N_s = 410$

FIG. 16.

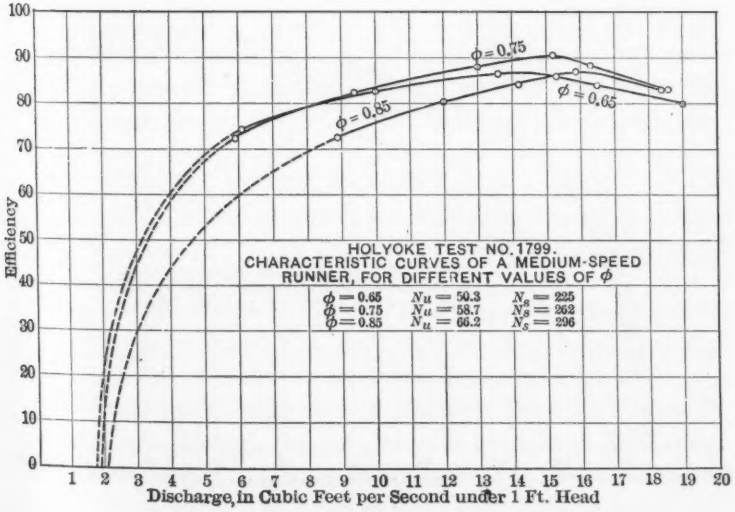

Efficiency
Discharge, in Cubic Feet per Second under 1 Ft. Head
$\phi = 0.75$
$\phi = 0.85$
$\phi = 0.65$
HOLYOKE TEST NO. 1799. CHARACTERISTIC CURVES OF A MEDIUM-SPEED RUNNER, FOR DIFFERENT VALUES OF ϕ
$\phi = 0.65$ $N_u = 50.3$ $N_s = 225$
$\phi = 0.75$ $N_u = 58.7$ $N_s = 262$
$\phi = 0.85$ $N_u = 66.2$ $N_s = 296$

FIG. 17.

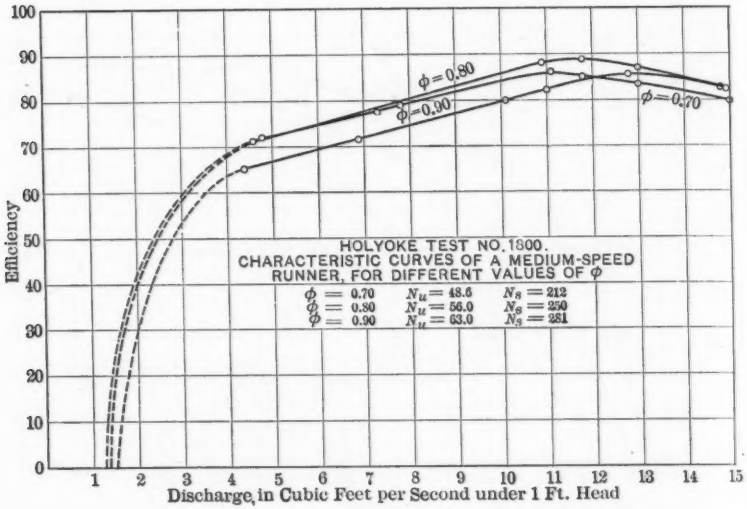

Efficiency
100
90
80
70
60
50
40
30
20
10
0
1 2 3 4 5 6 7 8 9 10 11 12 13 14 15
Discharge, in Cubic Feet per Second under 1 Ft. Head
$\phi = 0.80$
$\phi = 0.90$
$\phi = 0.70$
HOLYOKE TEST NO. 1800.
CHARACTERISTIC CURVES OF A MEDIUM-SPEED RUNNER, FOR DIFFERENT VALUES OF ϕ
$\phi = 0.70$ $N_u = 48.6$ $N_s = 212$
$\phi = 0.80$ $N_u = 56.0$ $N_s = 250$
$\phi = 0.90$ $N_u = 63.0$ $N_s = 281$

FIG. 18.

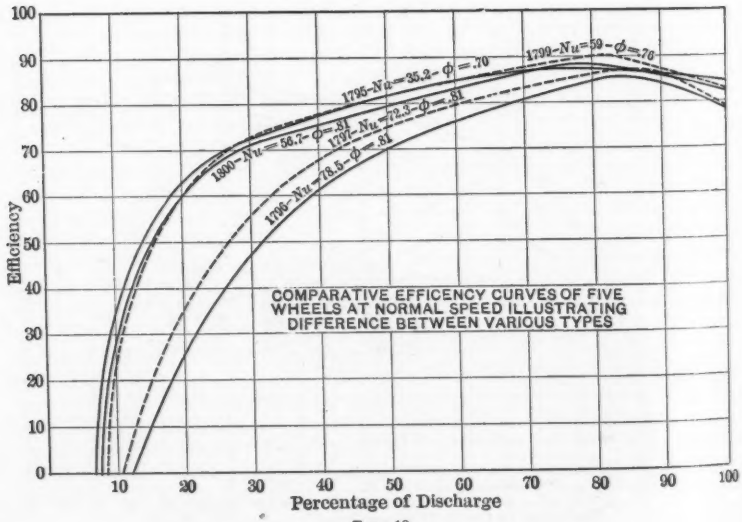

Efficiency
100
90
80
70
60
50
40
30
20
10
0
10 20 30 40 50 60 70 80 90 100
Percentage of Discharge
1795–N_u = 35.2– ϕ = .70
1799–N_u = 59 – ϕ = .70
1800–N_u = 56.7– ϕ = .81
1797–N_u = 72.3– ϕ = .81
1796–N_u = 78.5– ϕ = .81
COMPARATIVE EFFICENCY CURVES OF FIVE WHEELS AT NORMAL SPEED ILLUSTRATING DIFFERENCE BETWEEN VARIOUS TYPES

FIG. 19.

squares of the diameters, and that the speed varies inversely as the diameters, with constant efficiency.

Let D_a = the diameter of A,

D'_a = the diameter of a wheel of homologous design, having a capacity of 1 h.p.,

N'_a = the speed of the latter.

Then

$$D^2_a : D'^2_a = 2.07 : 1$$

$$D_a : D'_a = \sqrt{2.07} : 1$$

also

$$D_a : D'_a = N'_a : 50$$

then

$$\sqrt{2.07} : 1 = N'_a : 50$$

$$N'_a = 50\sqrt{2.07} = 72$$

Similarly $N'_b = 75$.

It is now plain that B is the higher-speed type, because, when both wheels are developing 1 h.p., B runs at 75 rev. per min., whereas A runs at only 72 rev. per min. As will be explained later, these are the "unit speeds" of these two wheels.

It should be clearly understood that this term, "high-speed," thus involves a consideration of power as well as speed. It is not related directly to the peripheral speed, although, as stated before, high peripheral speed is usually an attribute of a high-speed wheel. At the same time, it may also be an attribute of a low-speed wheel, and therefore it is not a proper criterion of speed in the sense in which it is here used. Wheel No. 1800 illustrates this point very well. At the normal speed of this wheel, the coefficient of peripheral speed, $\phi = 0.81$. This value is as high as for any of the other wheels, and yet this wheel is lower in speed than all the others, except No. 1795.

Unit Speed.—The "unit speed," N_u, used throughout this paper, is in all cases the speed, under 1 ft. head, of a wheel of the same design as the one under consideration, and of such size as to develop 1 h.p. at full gate. Other units might be used and serve the same purpose of comparison, but in America these are the most convenient. In Europe, under the metric system, the practice is to reduce to one metric horsepower, under 1 m. head. The result is termed the "specific speed," and is usually designated by the symbol, N_s.

The general equation for N_u is developed as follows:

N = speed of any runner,
P = power of same,
D = diameter of same,
h = head on same,
h_u = unit of head,
P_u = unit of power,
N_u = unit of speed,
N' = speed of above runner under unit head,
P' = power of above runner under unit head,
D_u = diameter of homologous runner developing unit power under unit head.

Then

$$N' : N = \sqrt{h_u} : \sqrt{h} \qquad N' = N\sqrt{\frac{h_u}{h}} \quad \ldots\ldots (a)$$

$$P' : P = h^{\frac{3}{2}}{}_u : h^{\frac{3}{2}} \qquad P' = P\left(\frac{h_u}{h}\right)^{\frac{3}{2}} \quad \ldots\ldots (b)$$

$$P' : P_u = D^2 : D_u{}^2 \qquad D_u = D\sqrt{\frac{P_u}{P'}} \quad \ldots\ldots (c)$$

$$\frac{N'}{N_u} = \frac{D_u}{D} \quad \ldots\ldots (d)$$

Substituting Equations a, b, and c in Equation d.

$$\frac{N\sqrt{\frac{h_u}{h}}}{N_u} = \frac{D\sqrt{\frac{P_u}{P\left(\frac{h_u}{h}\right)^{\frac{3}{2}}}}}{D} = \sqrt{\frac{P_u}{P\left(\frac{h_u}{h}\right)^{\frac{3}{2}}}}$$

Simplifying,

$$N_u = \frac{N \times \sqrt{P} \times h_u{}^{\frac{5}{4}}}{P_u \times h^{\frac{5}{4}}} \quad \ldots\ldots (1)$$

With $P_u = 1$ h.p. and $h_u = 1$ ft. this becomes

$$N_u = \frac{N \times \sqrt{P}}{h^{\frac{5}{4}}} \quad \ldots\ldots (2)$$

Given the power and speed of a wheel under any head, the unit speed may be determined by substituting in Equation 2. It is not

necessary to go through the process of double reduction as was done in comparing wheels A and B. For example, take wheel A, just considered.

Here

$$h = 100 \qquad P = 2\,080 \qquad N = 500$$

Substituting in Equation 2,

$$N_u = \frac{500 \times \sqrt{2\,080}}{100^{\frac{5}{4}}} = 72$$

which checks the former computation. Under the metric system, $P_u = 1.01$ h.p. and $h_u = 3.28$ ft. Substituting in Equation 1,

$$N_s = \frac{N \times \sqrt{P} \times 3.28^{\frac{5}{4}}}{1.01 \times h^{\frac{5}{4}}} = 4.46 \times \frac{N \times \sqrt{P}}{h^{\frac{5}{4}}} \ldots\ldots\ldots (3)$$

or

$$N_s = 4.46\, N_u.$$

Hence, to transform a unit speed from the English to the metric system, multiply N_u by the coefficient, 4.46.

This method of classifying wheels according to their unit speeds is one of great convenience. Suppose, for example, that it is desired to equip a plant with units using runners having a maximum capacity of 1 000 h.p., each at 200 rev. per min., under 40 ft. head. From Equation 2, $N_u = 63.3$, and therefore only wheels approximating this unit speed need be considered.

Again, suppose that the sizes of the units and the speed for a prospective plant under 100 ft. head have not been determined. It is desired to know what results could be obtained with a certain runner, the characteristics of which are well suited to the proposed service. The unit speed of this runner is 70. Substituting in Equation 2,

$$70 = \frac{N \times \sqrt{P}}{100^{\frac{5}{4}}} \qquad N \times \sqrt{P} = 22\,120.$$

This type of wheel will give any combination of power and speed, whatsoever, that will satisfy this equation. With the inverted scale of a duplex slide-rule, innumerable combinations of power and speed may be read off as rapidly as one can move the runner. This is a great convenience in preliminary calculations. In the above case, for

instance, any of the following combinations of power and speed can be produced, depending on the size of the runner used.

1 000	h.p.	at 700	rev.	per min.
1 460	"	at 600	"	
1 960	"	at 500	"	
3 060	"	at 400	"	
5 450	"	at 300	"	
12 200	"	at 200	"	
49 000	"	at 100	"	

Classification According to Unit Speeds.—Some method of classifying turbine runners on the basis of the duty which they are able to perform is essential to an intelligent study of turbine characteristics. It is obviously futile to compare the relative merits of two wheels which cannot accomplish the same work. They are in different classes, and, as such, might naturally be expected to possess different characteristics.

Since the unit speed is a measure of complete performance, both as to speed and power, it supplies an admirable basis for the desired classification. In this it should be noted that such a classification is independent of the details of design. The angles of the vanes, general proportions, peripheral speed, etc., may all be different, and yet the wheels are classified as of the same type if they have the same unit speed, because they, and they alone, are capable of doing the same work.

In Europe it is the practice to classify runners according to their coefficients of peripheral speed. A "low-speed" runner has a value of ϕ equal to about 0.6, a "normal-speed" runner, about 0.7 and a "high-speed" runner, about 0.8. These distinctions, however, embody the element of capacity as well as speed, although in an indirect and rather indefinite way. This is because a value of $\phi = 0.6$ is not ordinarily used, except with wheels of low capacity, 0.7 with those of normal capacity, and 0.8 with those of high capacity. The distinguishing element of capacity is really there, although its influence is not definitely measured. Such a method, of course, cannot be as exact as that based on unit speeds. In the latter case, both capacity and speed are definite mathematical factors.

Influence of ϕ Upon the Characteristic Curves.—The extent to which changes in the value of ϕ at normal speed affect the efficiency curves of turbine runners is difficult to determine, because, with wheels of rational design, the value of ϕ, as just stated, keeps step with the variation in the capacity of the wheel. It is, of course, a principle of economical design that a runner shall be made no larger in diameter than necessary. For a given speed, the lower the value of ϕ the smaller the wheel, and hence a low value of ϕ is suitable to a low-capacity wheel, and a high value to a wheel of large capacity. It may be generally stated as a principle of rational design that ϕ should always be made as low as possible, and that it should be forced up to high values only in order to get additional capacity.

Wheel No. 1800 was an experimental runner designed contrary to this principle for purposes of investigation. It is of irrational design in that it is a wheel of comparatively low capacity with a high value of ϕ. The value of ϕ for a wheel of this capacity need not be over 0.7 at normal speed, whereas it was 0.81, according to test.

The results obtained with this wheel appear to indicate that, with wheels of similar capacity, the value of ϕ at normal speed has no very important effect on the characteristic curves, although a high value of ϕ does show some tendency to pull down the efficiency. Fig. 19 shows that the three wheels of medium and low capacity gave similar results at normal speed, although the values of ϕ were 0.70, 0.76, and 0.81, respectively. The three wheels, however, which had the same value of ϕ, namely, 0.81, but differed considerably in capacity, ranging from medium power to the extreme limit of high power, gave widely different curves. All of which seems to indicate that forced capacity is the chief factor which causes the variation.

A comparison of Figs. 14, 17, and 18 shows a further similarity between the three wheels having different values of ϕ. The variation in efficiency at part gates, due to a variation of 0.1 in ϕ both above and below its normal value, is quite similar in all three cases. In fact, what difference there is between the three wheels is in favor of the high value $\phi = 0.81$. This wheel shows less drop in efficiency at part gates, due to an increase in speed, than either of the others.

In view of this evidence, and for lack of any experimental proof to the contrary, the writer believes it safe to say that the efficiency is not a function of ϕ, to any considerable extent.

Principal Difference Between Low-Speed and High-Speed Wheels.—Before proceeding further, it may be well to state that the deductions made from these tests should not be given too strict an application. Results necessarily vary with changes in design, and, while the results obtained with these wheels are of a nature broadly representative of other wheels of similar capacity, nevertheless, considerable variation in individual cases must be expected.

The most significant fact to be observed, in connection with the efficiency-speed curves of these wheels at various gates (Figs. 4, 6, 8, 10, and 12) is that all of them peak at different speeds. The most efficient speed of the wheel varies with the gate opening, increasing as the latter increases. This is true of all reaction turbines, whether of low or high speed, but not in the same degree. In the case of high-speed wheels, the difference between the best low-gate speed and the best full-gate speed is much greater than in the case of medium-speed and low-speed wheels. Table 6 shows the comparison between 0.4 gate and full gate for all the wheels under 1 ft. head, listed in the order of their capacity.

TABLE 6.

Wheel No.	1795	1800	1799	1797	1796
Most efficient full-gate speed	40	52	48	58	64
Most efficient 0.4-gate speed	31	41	38	35	35
Percentage of drop in speed	22.5	21.1	20.8	39.6	45.3
N_u	35.2	56.7	59.0	72.3	78.5

It may be observed from Table 6 that the drop for low-speed wheel No. 1795 and for the two medium-speed wheels, Nos. 1799 and 1800, is about the same, whereas it is nearly twice as great for the high-speed wheel No. 1797, and even greater for the extremely high-speed wheel No. 1796. This drop does not seem to vary much with the capacity of the wheel for values of N_u below 60, approximately. Above $N_u = 60$, or thereabouts, the drop increases rapidly as N_u increases.

When operating at constant speed, the inevitable result of this variation in the most efficient speed of the wheel is a sacrifice of efficiency at all gates except one. For example, if a speed is selected which will give the highest efficiency which it is possible to obtain at 0.75 gate, that speed will be too high to give the best obtainable efficiency at the lower gates and too low to give the best results at the higher gates. The best speed at which to run a wheel, therefore,

depends on which gate opening is the most important as regards efficiency. The best speed is the one which strikes the peak of the efficiency curve for that gate.

Naturally, the less variation there is in the best speed from low gate to full gate the less sacrifice of efficiency there will be when the wheel is run at constant speed. It has just been shown that this variation is much less with wheels of low and moderate unit speeds than with those of high unit speed, and therein lies the most important difference between these several types.

As an illustration of this point, compare low-speed wheel No. 1795 with high-speed wheel No. 1796 (Figs. 4 and 6). The maximum efficiency of the former at 0.444 gate is 82%, occurring at a speed of 34.5 rev. per min. The efficiency at the normal speed of 40 rev. per min., however, is 80%, and hence 2% has been sacrificed at this gate to the requirement of constant speed.

In the case of No. 1796, the maximum efficiency at 0.462 gate is 74.8%, occurring at 37.8 rev. per min. At normal speed of 53 rev. per min. the efficiency is only 62.5% and about 12% has been sacrificed to the requirement of constant speed. In addition to this, the peak of the curve of No. 1795 is about 7% higher than that of No. 1796, and hence the total difference in favor of the low-speed wheel at this gate is about 17 per cent. Fig. 19 shows graphically what a wide difference exists between the efficiency curves of medium-speed and high-speed wheels. The relatively poor showing of the two high-speed wheels, Nos. 1796 and 1797, is due to the two characteristic deficiencies just alluded to, namely, rapid drop of the efficiency-speed curves at part gate, and relatively lower maximum efficiencies at all gates. The latter condition is theoretically inevitable. It is impossible to discharge a large quantity of water through a wheel of given size as efficiently as a moderate quantity.

Another peculiarity which is distinctly characteristic of high-speed wheels is that the power at the high gates increases as the speed increases beyond normal. By over-gating the wheel it is possible to hold up a substantial increase of power with good efficiency at extremely high values of ϕ. For example, wheel No. 1796 (Fig. 6), at 1.077 gate, increases constantly in power and efficiency until ϕ has reached a value of 1.00. No. 1797 (Fig. 8), at 1.00 gate, increases until $\phi = 0.97$.

This feature of high-speed wheels is a valuable one, because it fits them especially for the only work for which they are at all adapted, namely, low heads. Low-head plants are almost invariably subject to relatively large reductions of head, due to back-water in the tail-race. If the head drops very much, the value of ϕ necessary to maintain constant speed will run up very high. A wheel which gives more power at a high value of ϕ than it does at the normal value, is a great advantage in such a case, because it increases the capacity of the turbine by just that much at a time when the power is perilously deficient on account of the reduced head. At such a time, it is necessary to open the turbine gates farther than would be advisable under the normal head. Under the latter, such over-gating would be wasteful, because the increase in power is very small and not at all porportionate to the increase in discharge. With some wheels, the power is even less than it was at the previous gate, on account of excessive drop in efficiency. Under the reduced head, however, with high values of ϕ, the increase in power is qute considerable. With No. 1796 (Fig. 6), for instance, the increase in power from 1.00 gate to 1.077 gate, at normal speed, is practically nothing, due to a 4% drop in efficiency. At $\phi = 1.00$, however, the increase in power is about 20%, and the efficiency is about 6% better. The over-gate, therefore, should be used only under a reduced head.

Reference to Figs. 4, 10, and 12 will show that the wheels of lower unit speed do not possess this characteristic. In every case, the high-gate curves fall off at the high values of ϕ.

Considerations Affecting the Choice of Wheels for Low Heads.— It is futile, of course, to attempt to define within fixed limits the heads which should be included in this classification. Some plants under moderately high heads operate under conditions very similar to those which are distinctly characteristic of the typical low-head plant. Others under low heads possess some of the characteristics common to high heads. If any useful deductions are to be drawn from such a study, they must be based on a consideration of typical conditions, with a liberal allowance for the inevitable exceptions which require special treatment.

A typical low-head plant may be illustrated by an installation where the head is created by building a dam across a large river flowing through a comparatively flat country. In such cases, it is im-

practicable to build a high dam, because the water cannot be confined behind it.

Having, probably, little storage area, other than the bed of the stream, such a power cannot usually be developed much beyond the minimum flow of the stream. If it were possible to impound the water which is not used during light loads for use during peak loads, then the part-gate efficiency of the turbines for such a plant would be of importance; but, usually, the water which is not used during light loads goes over the dam, and therefore the efficiency of the turbines at such times is relatively unimportant. Under such conditions, the poor part-gate efficiency of high-speed wheels is of no importance.

The important considerations are high efficiency at normal gate, as affecting the normal quantity of power which can be developed from the available flow; high speed, as affecting the cost of the electrical machinery and, to a limited extent, the cost of the turbines; high power at constant speed under reduced heads due to back-water, which is the inevitable coincident of flood conditions.

In respect to the first consideration, the high-speed wheel is somewhat deficient, depending on the extent to which the wheel is pushed for speed. Taking the efficiency of No. 1799, 90.43%, as the standard for a wheel of medium speed, the high-speed wheel No. 1797, yielding 87.39%, suffers about 3% by comparison. Wheel No. 1796, which is a more extreme type than No. 1797, gave 86.15%, or about 4¼% less.

Another point to be noted is that the maximum efficiency of high-speed wheels occurs at a higher gate opening than that of lower-speed wheels. With the latter it usually occurs at from 0.70 to 0.75 gate, whereas with the former it is usually from 0.80 to 0.85 gate. In order to provide for the customary overload of 25% with high-speed wheels, it is necessary that they shall carry the normal load at a lower gate than the most efficient one. This causes a further loss of efficiency.

In respect to the second and third considerations, the high-speed wheel has an overwhelming advantage, and is undoubtedly the best wheel for typical low-head service. Table 7 gives a comparison of the results which could be accomplished by the three types of runner, Nos. 1796, 1797, and 1799, respectively, if applied to a plant having a maximum capacity of 500 h.p. per runner under 20 ft. head. Low head is assumed to be 13 ft.

TABLE 7.

Wheel. No.	20 Feet.			13 Feet.			N_u
	Horse-power.	Revolutions per minute.	Efficiency.	Horse-power.	Revolutions per minute.	Efficiency.	
1796	500	9	82.0	279	149	80.0	78.5
1797	500	6	83.0	277	136	80.5	72.3
1799	500	2	82.7	246	112	79.5	59.0

Runners for Intermediate Heads.—It is as the head increases and back-water conditions become less important, and as the speed of the ordinary runner, increasing with the head, more nearly approaches an economical value, that the use of the high-speed runner becomes more questionable. As these conditions change, the considerations which favor high-speed wheels gradually disappear, and others of more importance become dominant.

As the head increases the storage capacity usually increases, and, with the capacity to impound water for peak loads, the part-load economy of the turbines becomes highly important, and the medium-speed wheel with its high efficiency is clearly indicated. The higher the head the more valuable a cubic foot of water becomes, and the more important it is to save it. High speed becomes a subordinate consideration. If the additional power which can be developed by the lower-speed wheels with their high efficiency is worth more than the interest on the saving in electrical apparatus which would result from the use of high-speed wheels, then the former are certainly the more economical proposition. This principle, of course, is fundamental and commonly recognized, but is not always intelligently applied for lack of specific knowledge regarding the disadvantages incidental to high speed. If purchasers were as well informed as they ought to be, as to the deficiencies of high-speed runners, there would be fewer of them in use at the present time. It is hoped that this paper may supply some welcome information.

Table 8 shows comparative efficiencies at various gates for medium-speed wheel No. 1799 and high-speed wheel No. 1796. In view of these figures, a very weighty reason would be required to justify the use of No. 1796 instead of No. 1799, if efficiency is of any importance.

High-Head Wheels.—Runners for high heads pass through all the intermediate stages of design between the mixed-flow wheel and the

purely inward-flow or so-called Francis type, according to the head available and the power developed. As the head increases the unit speed tends to decrease, and the design approaches that of the low-capacity Francis type (No. 1795). Although the designs of these wheels vary considerably, the characteristic curves vary but slightly from the best results of medium-speed wheels.

TABLE 8.

Wheel No.	Gate Opening.									
	0.30	0.40	0.50	0.60	0.70	0.75	0.80	0.85	0.90	1.00
1799	68.5	76.5	82.0	86.0	88.0	90.4	90.0	89.2	88.0	83.0
1796	41.0	55.5	66.0	73.5	79.5	82.0	84.5	86.2	85.0	82.0

The efficiency curves of wheel No. 1795 (Fig. 5) at normal speed, with $N_u = 35.2$, and wheel No. 1799 (Fig. 11), with $N_u = 59$, are very similar. On account of the discrepancy between the peaks of these two curves, the part-gate efficiencies are about the same (Fig. 19). Otherwise, the latter would be a trifle higher for the low-speed wheel. The general tendency is for the efficiency, particularly at low gates, to improve as N_u decreases.

The reason wheel No. 1795 did not show as high maximum efficiency as No. 1799 is that, being a low-speed wheel, it developed much less power under test, and hence the loss, due to mechanical friction, was a much greater percentage of the total power developed than was the case with No. 1799. Wheel No. 1795, operating under a head of several hundred feet, particularly in large diameters, would undoubtedly develop 90% efficiency or better. The effect of mechanical friction, which has been discussed at length, is especially detrimental to the efficiency of a low-power wheel such as this, when tested at Holyoke. No high-head wheel can by any possibility show as high efficiency in the Holyoke flume as it would in regular operation, assuming, of course, that in the latter case it is properly installed.

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PAPERS AND DISCUSSIONS

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SOME EXTENSIVE RAILROAD SURVEYS, AND THEIR COST PER MILE.

Discussion.*

BY MESSRS. CLINTON S. BISSELL, F. LAVIS, E. W. LEWIS, AND GEORGE L. DILLMAN.

Mr. Bissell.

CLINTON S. BISSELL, M. AM. SOC. C. E. (by letter).—The author states specifically that the paper is not a theoretical discussion of railroad location, and later he cites as the "desired results":

1.—Easiest grades possible, against west-bound traffic;

2.—Lightest curvature;

3.—Shortest line, etc.

He also mentions ruling grades of 0.3% on the Little Kanawha Division, and 0.5% on the Burnsville and Eastern Division. The momentum grade at Mile 20 on the latter division, is introduced "to avoid a long détour"; and, in fact, the reader is impressed throughout the paper by the evident great advantage of the "shortest line."

To the writer, a railroad is simply an industrial plant, instituted to furnish transportation, and the "desired results" to be realized, are embodied in the one problem, viz., to obtain the line of greatest efficiency. The author has mentioned no measure of efficiency, nor has he indicated any limitations as to the duty required of the line. Without these, the writer is at a loss to understand the basis upon which the location was made, unless it was merely to get a tolerably direct line, with gradients as low as good direction would allow. The author has either disregarded or withheld one element which Wellington

* This discussion (of the paper by W. S. McFetridge, M. Am. Soc. C. E., printed in *Proceedings* for April, 1909, and presented at the meeting of May 19th, 1909), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

rightly cites* as of "overwhelming importance," namely, the probable amount of traffic. The second important element—that of easy gradients—the author has observed without correlation to the probable traffic. The other two elements—curvature and distance—are, within reasonable limits, minor details of location, and yet apparently the author has given them great attention, to the negligence of one of the two more important elements. Mr. Bissell.

Since nothing can well be further removed from the theoretical side of the problem of location than the actual dollars and cents which the road can either earn or save, let us consider such part of the profile as the author has shown in Fig. 3, between Burnsville and Buckhannon.

From Buckhannon westward it is noticeable that the two stretches of ruling 0.5% grade amount to about 3 miles in 33, or only 10% of the distance, neglecting the momentum grade which, according to the author, can be removed.

Assuming at random that the line will carry annually 3 000 000 tons of freight westward, with heavy locomotives and modern high-efficiency cars, the 0.5% ruling grade permits a train load of 1 700 tons of lading; and the same train, returning with about one-third as much lading, can surmount the 1% grades going east. Taking the train-mile cost at $1, for convenience, the annual cost of operation can be roughly estimated.

It is rather surprising that the two ruling 0.5% grades were not made 0.3% to accord with those on the Little Kanawha Division. For the sake of example, it will be assumed that this could have been done at the expense of 4 miles more of distance at a gross cost of $50 000 per mile. For the 0.3% grade the train load going west would be 2 420 tons of lading; and the same train would return east with about one-fifth as much lading, against the 1% grades. Hence, the following figures may be deduced:

3 000 000 ÷ 1 700 × 33 × $1	.....west-bound	$58 240
Same trains returning	.....east-bound	58 240
Annual operating expenses on 0.5% grades		$116 480
3 000 000 ÷ 2 420 × 37 × $1	.....west-bound	$45 870
Same trains returning	.....east-bound	45 870
Annual operating expenses on 0.3% grades		$91 740
Difference		$24 740
Less interest at 4% on cost of 4 miles of line		8 000
Annual saving		$16 740

* "The Economic Theory of Railway Location," by the late A. M. Wellington, M. Am. Soc. C. E.

Mr. Bissell. At 4%, this saving justifies a present expenditure of $418 500 to reduce the ruling grades from 0.5% to 0.3%; this reduction increases the efficiency of the line by a saving of 14% in expenses; and probably there would be enough capital left to take out the momentum grade at Mile 20.

Unless the matter of location be viewed from some such standpoint as this, the writer does not understand how the "shortest line" practicable on the "easiest grades possible" and on the "lighest curvature" can be decided. The intimate correlation of these three elements does not permit of such a decision, except upon some basis other than simply the contour of the country.

A few years ago the writer made several surveys and locations along the south shore of Nova Scotia between Cape Sable and Shelburne. The drainage, flowing down to the sea, crossed the direction of survey about at right angles. The route along the seashore was too irregular to justify a low line for any considerable distance, and a high line several miles inland touched but few towns, because these, for the most part, were situated along the shore. The final finished location was a compromise between these two lines. The whole route was wooded, especially on the slopes of the ridges where the maximum grades occurred. The specifications of the Dominion and the Provincial Governments fixed the maximum grade at 1.5% and the maximum curvature at 6 degrees.

In conducting these surveys, the writer found it necessary to take contours only on the ruling grades and a few other points on the line. The grades were first cut through, and the profile taken, after the manner mentioned in this paper, from the summit downward. To accomplish this object, the transit was frequently carried several miles ahead of the measured line and set in the summit or other governing point, and the profile run from a temporary bench-mark, the elevation of which was assumed or estimated with a barometer. The measured line was then brought forward, connected, and run again over the maximum grade. Particular attention was then given to the contours and to the topography of the stream crossing which was usually found at the foot of the grade. The great advantage in this method is the fixing of the proper point of connection at the foot of the grade with the assurance of passing through the governing point ahead. The writer believes that notwithstanding the necessity of running over the grade twice, time is saved by avoiding much backing up and subsequent alteration of the line back of the foot of the grade. It gives a continuous chainage, saves much contouring, and produces a preliminary map which lies close to the location at all the critical points.

The author's figures on costs are very interesting and valuable; but

the maximum rate of progress mentioned, 7 miles of preliminary line and 4½ miles of location per day, has never been equaled by the writer. He considers it excellent work for one party to accomplish even half of these distances in one day. Mr. Bissell.

F. LAVIS, M. AM. SOC. C. E.—This paper is a valuable addition to the somewhat scanty literature relating to the subject, and, undoubtedly, will be of considerable interest to those connected in any way with the conduct of surveys for the location of railroads. It is especially so to the speaker, as the methods used agree very closely with those developed by him between 1898 and 1902 on the Choctaw, Oklahoma and Gulf Railroad.* Those surveys resulted in the construction of more than 800 miles of railroad, and, probably, at least as many more miles of location were actually staked out on lines which were not built. The speaker has no exact figures at hand, but it is probable that more than 5 000 miles of preliminary were run during that time. Mr. Lavis.

During the last ten or fifteen years these general methods, with slight minor variations to suit local conditions and individual ideas, have become fairly well recognized as standard on most of the larger railroad systems of the country; at any rate, those engaged to any extent in building new lines and extensions, recognize the absolute necessity of such methods, in order to determine without doubt that the line finally adopted is such "that no other line can be built through the same country, with the same or better ruling grades, with less expenditure, at the same unit prices."†

The data relating to costs show close agreement with the costs presented by the speaker in the paper referred to, and, although, in both instances, they are somewhat higher than a great many locating engineers think their surveys have cost, it will generally be found that, when these low costs are given, the fact has been lost sight of, that on any survey for the location of a railroad, the costs, both of the field parties, and a proper proportion of the expenses of the main and division offices, covering all work from the reconnaissance to the completion of the final maps and profiles, ready for the contractors to bid on the construction, and the real estate agents to purchase right of way, should be included.

The surveys described by Mr. McFetridge were made in 1902 and later; the costs previously given by the speaker were for surveys made during the latter half of 1902, so that the time agrees fairly closely. During 1903, of course, it was easier to get men; and salaries, as a rule, were comparatively low, so that the costs, during the latter part of the time when these surveys on the Little Kanawha were made,

* *Transactions*, Am. Soc. C. E., Vol. LIV, p. 104 (1905).

† "Railroad Location Surveys and Estimates," p. 8.

Mr. Lavis. would tend to be low rather than otherwise; this is shown also by the salary list.

Comparing the costs with those presented by the speaker,* it may be noted that the costs of the field work of the preliminary and location lines agree very closely. The average cost of the lines, as given in Table 2, shows the preliminary to have been about $28, and the location $88 per mile, whereas the costs given by the speaker were $26 and $71, respectively. The costs of the completed location, as described in the paper, vary from $127 to $188 per mile, averaging $151, on the basis of the mileage of located line staked out; or from $171 to $251, averaging $202, if the mileage of adopted location be taken, which latter may be compared with the cost of $192 per mile of adopted location as given by the speaker. The costs of field parties per day are given by Mr. McFetridge as between $35 and $40, whereas those described by the speaker cost from $42 to $65, and averaged $46. The somewhat lower cost per day of the field parties on the Little Kanawha, although the cost per mile of line was higher, is probably due to the difference in organization. On the Choctaw, although all lines were subject to revision, of course, by the Chief Assistant Engineer and the Chief Engineer, the Locating Engineers, whose whole expense, in addition to that of an Assistant Locator, was charged to the field party, were expected to be able to decide questions which, on the Little Kanawha Railroad, were apparently not left to be settled by the Assistant Engineer in charge in the field.

Judging from the description of the country and the number and length of preliminary lines run, it would appear that the two surveys are fairly comparable. The country described by Mr. McFetridge is in many cases rougher in detail than that through which the surveys described by the speaker were made, though, in the latter case, the character of the country required the detailed examination of a much wider range in order to determine the best location. This is shown by a comparison of the mileage of preliminary with that of location; on the Little Kanawha lines there were 2.37 miles of preliminary to 1 mile of location, whereas on the lines described by the speaker, the average was 3.14, a little greater than 30% more. The effect of the rugged detail is shown in the considerably increased cost of staking out the located lines on the Little Kanawha.

The generally prevailing idea as to the cost of railroad surveys is, that in the United States it should be about $100 per mile; and in a review† of the speaker's "Railroad Location Surveys and Estimates," a well-known railroad engineer made the statement that "much excellent location has been made in America in not easy country for less than $50 per mile of located line, and this cost includes the needed pre-

* *Transactions*, Am. Soc. C. E., Vol. LIV, p. 133.
† *Technical Literature*, April, 1907, p. 175.

liminary for that located line." He adds, referring to the supposed high costs of the Choctaw surveys, "We must conclude * * * that the parties were not made up of veterans." Veterans or not, the speaker does not believe that any line, outside of a tangent across a prairie, and sometimes not even that, can be located and proper maps completed, for any such sum as he mentions, and that for any ordinary country a rough estimate of $200 per mile would be much nearer correct for properly conducted surveys and the preparation of a proper set of plans ready for construction, these latter, of course, not including details of structures. The trouble with most of the published costs of location surveys is that they do not include anything but the cost of staking out a location on the ground; and supervision, office expenses, equipment, costs of preparation of proper maps, profiles, etc., are omitted. Mr. Lavis.

There is considerable difference, also, in the amount and kind of information developed by surveys. As an instance of a not uncommon form presented to a contractor as the location profile, Fig. 12 shows a part of a profile of a line in the Southwest, which was actually sent to a New York contractor, and on which he was asked to make his bid, this bid to be a lump sum for the whole line. Fig. 13 shows a section of a typical profile of located lines such as is usually required by most of the railroads in the West. The material in the cuts is explored by making test pits and soundings, as are also the foundations for structures, when necessary, and the information obtained is shown on the profile. The quantities in cuts and fills are given, with the approximate length and location of the haul; the quantities in structures are estimated and shown. In short, the profile contains all the information necessary to prepare a close estimate of cost, and gives the contractor all the information, as to the nature of the work, which he requires in order to make an intelligent bid.

The profile shown by Mr. McFetridge in Fig. 9, as the standard form for a location profile, seems to the speaker to be defective, inasmuch as it shows no quantities, gives little indication of the material to be excavated, or the amount or direction of haul, and no indication of the form or location of bridges, culverts, or other structures, or the quantities of material required to build them. The information required for the proper design of the larger bridges is shown in good shape on the situation plan, Fig. 10, but it is very desirable to show on the profile the type of structure and the space it occupies.

The speaker believes it to be of the greatest importance, not only to show the distribution of quantities on the location profile, but also to insist on their being worked out on the profiles of all projected locations. There is nothing which gives a man as completely the intimate knowledge of the line which it is really necessary for him to have as to work out the distribution, and get a clear idea of what he is to do

Mr. Lavis.

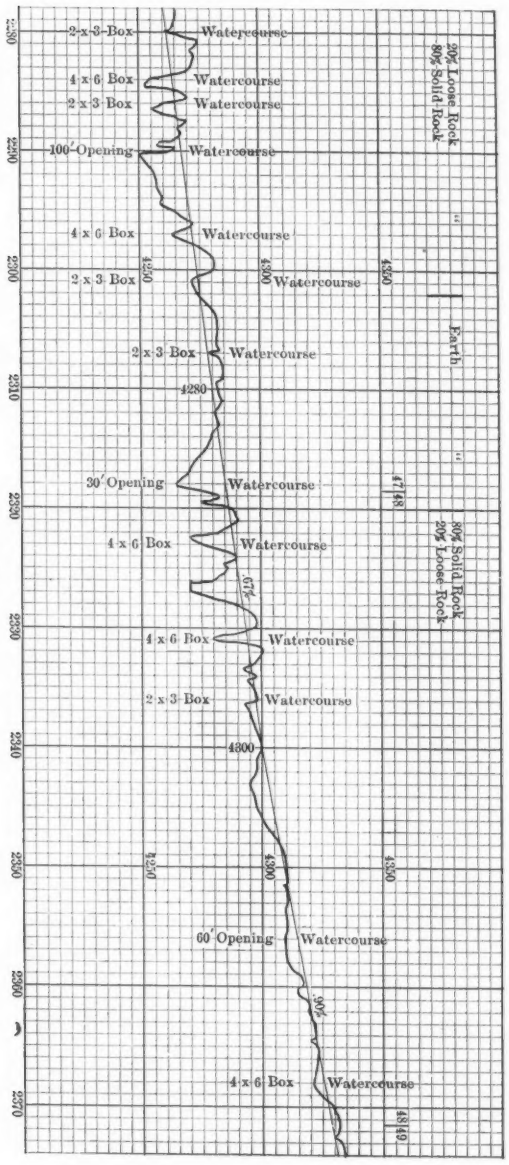

FIG. 12.

Mr. Lavis.

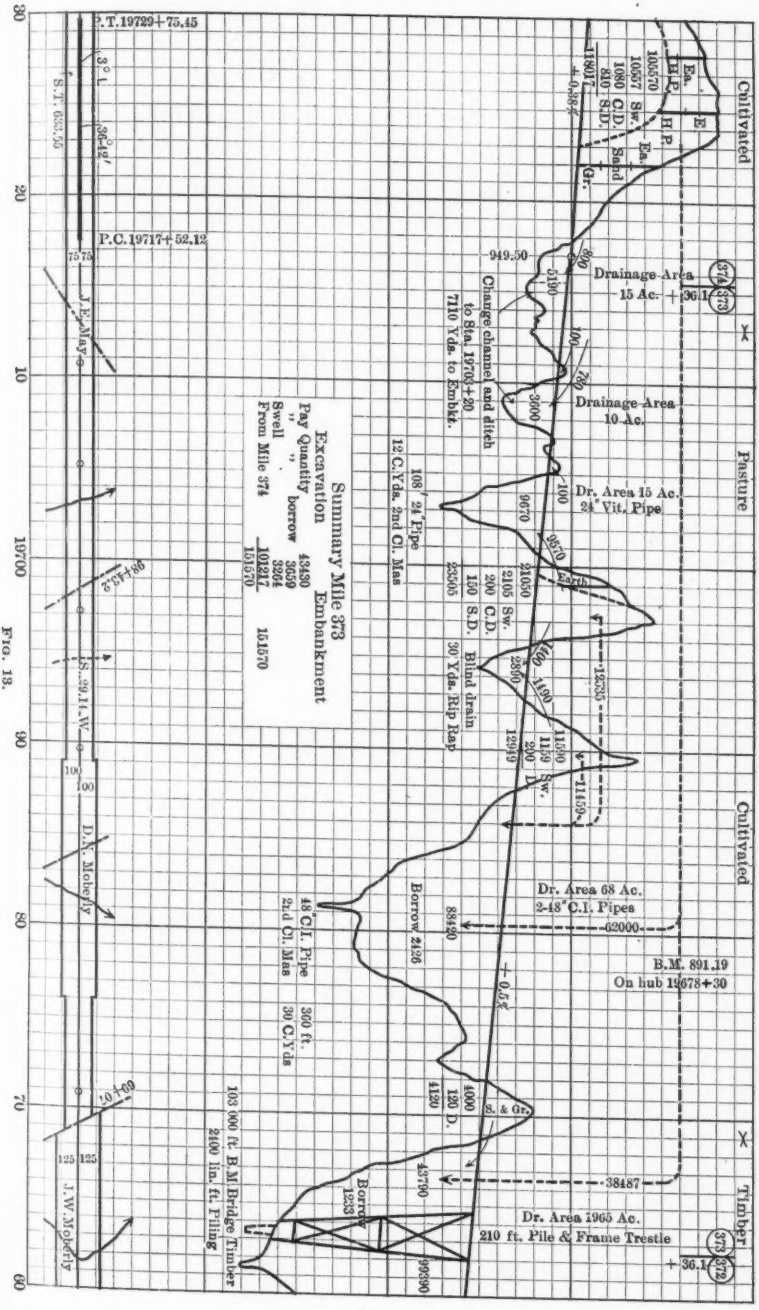

FIG. 13.

Mr. Lavis. with the excavation, and where he is to get material for the embankments.

One very important rule in regard to the final location, the speaker believes, should be made inflexible: After the profile of the located line has been platted, a temporary grade line fixed, tentative determinations of bridges, culverts, etc., made, and preliminary quantities calculated and distributed, this latter being most important, the Locating Engineer should take the profile into the field, and, walking over the line, should study carefully the situation on the ground in the light of the information which he then has, which, by that time, should be almost complete. This final review, if conscientiously done by a competent man, will often save many thousands of dollars in the construction, and avoid costly mistakes.

Although, in general, the methods described by Mr. McFetridge correspond closely to those described in the speaker's paper, previously referred to, a few minor differences may be noted. In keeping level notes the speaker prefers to keep the rod readings on the turning points entirely separate from the rod readings for elevations. It seems hardly necessary to have columns for grade and cut-and-fill in the location level notes, these belong more properly in the level notebook of the Resident Engineer, in which he records the check levels run on assuming charge of his residency.

The titles of profiles are generally more convenient when placed on the outside at each end; even if put inside they should be at each end rather than in the upper left-hand corner. Profiles accumulate rapidly on location, and with the title inside and only at one end, the chances are that two-thirds of them will have to be entirely unrolled to find the one sought.

The question of the advantages of separate sheets as compared with rolled maps has been discussed too often for the speaker to hope to say anything new at this time. Mr. McFetridge, however, although he evidently prefers the separate-sheet method, recognizes the necessity of having a large rolled map of the whole line in order to get a comprehensive view of the whole situation. The speaker takes exception to his very positive statement that the separate-sheet method of obtaining topography is quicker and better than any other; to say the least, this is open to argument. The speaker, who has had some little experience, prefers the method described in his paper.

For small general maps of the country, on which—in addition to the principal roads, trails, streams, etc.—may be shown the various preliminary lines as well as the final location, the speaker prefers a uniform scale of 5 000 ft. to the inch. These maps are useful, not only to the field parties and in the office of the Chief Engineer during the location, but also in discussions and conferences with the officials of the road, and others who only require general information. They are also

of convenience to contractors in bidding on the work, in order to enable them to form a better idea of its accessibility, etc. They should always be prepared at the beginning of the survey, and information should be added as the survey progresses. Mr. Lavis.

The speaker hardly sees the necessity of staking out a location for the purpose of choosing between two apparently equal routes, or, as was done in this case, for comparing the lines on either side of the larger watercourses. If the preliminary lines have been run with any degree of skill, and the topography has been properly taken, there should be no difficulty in getting, from a projected location on the map, and an inspection of the ground where this projected line would lie, all the information necessary for the comparison of any line or lines, except under the most exceptional circumstances.

The salaries named seem to be somewhat lower than those usually paid. In the West it is not uncommon to have a locating engineer in charge, with an assistant who looks after the details of the field work, leaving the former free to explore the country thoroughly, and devote more attention to the office work and the projection of various locations than he otherwise could. With a conscientious locating engineer this is a very good arrangement, and allows a much more careful threshing out of the country than would otherwise be possible. The author states that, on the surveys described:

"The Assistant Engineer was not expected to spend all his time with the party, but to be with it enough to see that work was going on satisfactorily, and that lines were being run over the proper routes; the remainder of his time was spent in a thorough study of the country, picking out routes to be examined, and looking after his necessary office work and correspondence."

The man who is responsible for the work of the party in the field should be with it most of the time, and if this devolves on the man in charge, it keeps his nose too close to the grindstone to allow of a proper perspective from which to view the problem as a whole, to say nothing of taking up practically his whole time. A proper, careful, study of the many possible projections cannot be made at night by a man who has put in already 10 or 12 hours' hard work in the field, although it is really surprising how much work men on locating parties manage to get through and thrive on.

The salary of the topographer is given at $65 per month, and, without intending in any way to cast any reflections on the young men who filled these positions, this seems to be altogether too little for good men. Taking topography has always seemed to the speaker to be a very simple operation, and it really is, if one has any sort of instinct for it, but it is remarkable how few men seem to grasp the idea, even after considerable experience. The speaker believes the topographer should be paid only slightly less than the transitman.

Mr. Lavis.

The fundamental idea of the proper principles governing the staking out of the located line is stated very concisely by the author in the third paragraph on page 320,* and this idea cannot be impressed too strongly on all connected with work of this class. "In staking out the location, the aim was to get a profile to correspond with the projection, and not to get the lines in exactly the same relative positions shown by the projections." In other words, there is to be no slavish following of the projected location as a line on the map, but rather to reproduce, on the ground, the line which the studies of the map and the general situation have shown is there, and is the best.

As the speaker has already pointed out elsewhere,† it is just here that modern methods differ from the old. It is neither the old paper location, made without reference to conditions in the field, nor the work of the Genius, described by Mr. Whinery‡ in his discussion of the speaker's paper, who threw his curves at the hills, as the cowboy throws his rope at a steer, and sometimes hit them and sometimes missed. If he missed, so much the worse for the railroad. The modern method collects all the available data affecting the location, eliminates definitely the unavailable lines, projects on the map the line which best fulfills all the requirements of the situation, and then uses all the intelligence of the Genius in reproducing that line on the ground. No extraordinary mathematical calculations are required to lay out the located line from the projection, only plain common sense and locating skill.

No amount of accuracy in making the original surveys and platting the map can compensate for lack of intelligence in transferring the projection to the ground, and extreme refinement of accuracy is wasted, if proper intelligence is used.

The statement on page 314,* contains, in a very small compass, the essence of the requirements which should govern the conduct of all location surveys, a "most minute study of the land, a thorough knowledge of its character, the running of many lines, the comparison of many estimates," are absolutely necessary in every case. The Locating Engineer, and by that is meant the man really responsible for the location, should be thoroughly steeped in the details of the country, and the word steeped is used advisedly, in the sense that there should be time enough in all cases to allow the information acquired to be taken in, digested, and to permeate him through and through.

One other point brought out in the paper, which is most important, is that actual surveys of all possible routes were made; as the author points out, this does not mean that an indiscriminate topographical map of the whole country was made, or was necessary. In spite of all that has been written about eliminating certain routes by eye, and per-

* *Proceedings*, Am. Soc. C. E., for April, 1909.
† "Railroad Location Surveys and Estimates," p. 125.
‡ *Transactions*, Am. Soc. C. E., Vol. LIV, p. 143.

haps by a hand-level, there is no question that absolute proof, by actual survey, should be furnished to the people putting up the money, or, at any rate, to those responsible for its expenditure, that there is no doubt that the location adopted is the best possible under the given governing conditions. The ability and so-called "eye for country," ascribed to the older generation of locating engineers, which many of them undoubtedly possessed, are just as necessary to-day, but must be supplemented by scientific methods, adequate surveys, and much hard work; and this is necessary, not only in order to be able to furnish proof that the line adopted is the best, but generally in order to find this line. As stated also in the paragraph referred to, "On a first examination of this country one was inclined to say offhand that it was impracticable to leave the river * * *; but * * * a thorough study * * * gave the question a different aspect." This experience has been duplicated many, many times, and in many cases the study was not made, and the railroad paid the bill later. Mr. Lavis.

Not many years ago it was not an uncommon thing to hear some of the Geniuses with "eye for country"—and not much of anything else —talk about the foolishness of trying to get a 0.5% line through a 1.0% country. The results obtained by these surveys are only another instance of the possibility and practicability of finding these lines, if one knows how to look for them. This description, of work actually accomplished, should be borne in mind by all who have to obtain money for such surveys, and, when necessary, should serve to stiffen their backbone to demand all that is necessary to carry out such surveys in an adequate manner.

The author is to be congratulated on the very clear and concise manner in which the information is presented, and his recognition and presentation of the many vital points which differentiate proper methods of making location surveys and getting results from the methods too often adopted, of simply stringing together a lot of tangents and curves over which a train can be run somehow or other, if the stockholders' money holds out long enough and no one else builds a line alongside over which trains can be operated at half the expense.

E. W. Lewis, M. Am. Soc. C. E.—This excellent paper is of much interest to engineers engaged in railroad location. The methods outlined by the author are very nearly the same as those used by the speaker in making many surveys in widely scattered sections of the country. The make-up of field parties is identical with that used by him on the Northern Pacific Railway, and quite similar to general practice. Mr. Lewis.

The method of plotting topography in the field, while advantageous in many respects, would be a difficult performance in a rainy country, such as the Pacific Northwest, where it rains almost every day for long periods during the winter season.

Mr. Lewis. The speaker prefers taking the notes in a topography field book. These books being ruled like cross-section paper, the contours can be plotted and the exact measured distance from the center line given for each 5- or 10-ft. contour, making it very easy to plot on a large map in the office. This plotting may be done in the evening, but, ordinarily, it is left until the next day. With a draftsman regularly employed to keep up the map, each day's field work may be mapped on the succeeding day.

The method of taking the contours, by measuring from the center line with a tape, and obtaining the difference of elevation with a hand-level, is in accord with the speaker's practice as being the best when an accurate contour map is desired. He has learned by sad experience that the saving of a little time in taking topography by pacing distances, etc., is often the cause of a much greater loss of time when the paper location is run out.

In regard to spiraling curves, if the line is likely to be a final location, particularly if the prospect is good for immediate construction, the speaker prefers to run in the spirals with the location, although, unquestionably, it saves considerable time not to do so.

The costs of surveys given in the paper, are interesting as showing the wide differences on various lines. The approximate costs of some surveys with which the speaker has been connected in various parts of the country may be of interest. On the north bank of the Columbia River, in Washington, over very rough, difficult ground, largely on steep slopes, with numerous short tunnels, where, in some instances, it was necessary to lower men down the cliffs with ropes in order to get the line staked, the cost for a stretch of 32 miles was about $325 per mile. In Massachusetts, through an average country, largely wooded, the cost of 20 miles of location was about $150 per mile. In North Dakota, 60 miles of location over a rolling prairie, and crossing one deep, broad valley, cost $110 per mile. In another section of the same State, over rolling prairie, the cost was $80 per mile. These figures are for the total cost of the final location, including all preliminary, trial, and abandoned lines.

Mr. Dillman. GEORGE L. DILLMAN, M. AM. SOC. C. E. (by letter).—The writer is impressed by the absolute lack of value of this paper to the Profession, the uselessness of the conclusions drawn, and the pernicious possibility of these conclusions being applied to some other case of railroad location. No two locations are alike, and hardly any two are comparable in time, cost, or value.

The writer was transitman on two locations which are worth noting as being extremes in his practice: The first was in Cottonwood Cañon, on the Grand River in Colorado, between Dotsero and Glenwood Springs. Afterward the Denver and Rio Grande built its line on the opposite side of the cañon. There was a great deal of clearing, and

the topography was such that direct measurement was often impossible, so that a great deal of triangulation for distance was necessary. This location cost more than $600 per mile. Mr. Dillman.

The second location was on the North Platte River, in Nebraska, extending more than 100 miles northwest from North Platte. The country was open, there was no clearing, the tangents averaged about 9 miles, and the curve angles between them were small. This location cost between $17 and $18 per mile.

These locations were each fully staked on the ground. The costs include maps, profiles, and computed estimates, as well as all preliminary work. The parties had about the same number of men and equipment, except that more axe-men were added to the Grand River party. Each party worked efficiently and sometimes for long hours. If there was any difference, the cheaper location was the more carefully made. Tracings of each map were filed with the Interior Department, accompanying applications for rights of way across Government land, and each location was ready for the construction crews.

These locations are probably not extremes; there may have been cheaper locations made; undoubtedly, there have been more expensive ones. What is desired is to bring out the fact that there is no possible connection between the costs of railroad locations; and that a full knowledge of the cost in one case cannot be applied to any other case.

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AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

CAISSON DISEASE AND ITS PREVENTION.
Discussion.*

BY MESSRS. J. S. HALDANE, FREDERICK L. KEAYS, ALBERT J. LOOMIS, WILLIAM L. SAUNDERS, WALTON I. AIMS, AND SEWARD ERDMAN.

Dr. Haldane.

DR. J. S. HALDANE, F. R. S. (by letter).—After very carefully reading Mr. Japp's paper, the writer considers it to be an extremely valuable contribution to a practical solution of the prevention of caisson disease. He clearly points out the difficulties involved in adopting without modification for all compressed-air work the plan of "stage decompression" worked out by Dr. Boycott, Lieutenant Damant, and the writer, which has been in successful use since 1907, for divers of the British Navy. A full account of the experimental evidence on which this plan is based will be found in the *Journal of Hygiene*, 1908, pp. 342-443. It will be seen from this paper that most of the experiments were made at very high pressures, such as 75 lb., and that they were carried out with a view to deep diving. It was for many reasons desirable to leave a good margin of safety; and, therefore, this margin was left, particularly as practical experience in deep diving was very limited, on account of other causes which the writer and his associates had just succeeded in removing.

Work in tunnels and caissons, however, is conducted at much lower pressures; practical experience is much more wide; and, thanks mainly to Mr. Moir, recompression is usually readily available. It becomes a question, therefore, whether the margin of safety which was allowed in decompression cannot be narrowed without appreciable danger. The new evidence which Mr. Japp has brought forward certainly seems to

*This discussion (of the paper by Henry Japp, M. Am. Soc. C. E., printed in *Proceedings* for April, 1909, and presented at the meeting of June 2d, 1909), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

indicate that this is the case. He records the fact that in the construc- Dr. Haldane.
tion of the East River Tunnels 330 "seasoned" men were employed for 36 days in two 3-hour shifts at a pressure of 40 lb., and were decompressed by a modified form of stage-decompression in 48 min., without a single serious case of caisson disease occurring. Thus there must have been about 24 000 individual decompressions without any mishap. On the principles laid down in the writer's paper, Mr. Japp calculates that the maximum air-saturation left in any part of the bodies of these men at the end of decompression, corresponded to a pressure of 27 lb. The writer's calculations agree with this estimate. In view of these results the author proposes to shorten stage-decompression in such a way that in place of the 19 lb. of residual saturation which the writer has proposed, 25 lb. should be left in the case of men who are "seasoned" to the work. This would greatly shorten stage-decompression, and render it a much more easily practicable process, although the pains in joints, etc., which are the most easily produced and least serious symptoms of caisson disease, would not be prevented.

The reason for the proposed 19 lb. is that occasional serious cases seem to occur at working pressures down to 20 lb., and that a fatal case is recorded at 23 lb. On the other hand, there can be little doubt that for the great majority of men a pressure of 25 lb. would not cause appreciable danger. Provided, therefore, that susceptible men can be excluded, Mr. Japp's proposal would seem to be consistent with reasonable safety.

Susceptible individuals can doubtless be excluded, to some extent, by a careful medical examination, with special regard to the state of the heart and blood vessels, and to any general debilitating cause, among which age must be included. Another very important factor is fatness. Dr. Boycott and Lieutenant Damant* have recently shown by experiments on animals that when the exposure to compressed air is prolonged, fatness increases the susceptibility to caisson disease. They decompressed 70 guinea pigs at a dangerous rate after exposure to a pressure of 100 lb. Exactly half of the animals died of caisson disease, and the other half were killed. The percentage of fat was then determined in the bodies of the two sets of animals, and was found to be much greater in those which had died of caisson disease, the average difference being in the proportion of 225 to 100. The caisson disease simply picked out the fatter animals. In consequence of these experiments, which confirmed in a most striking manner the conclusions which had been previously reached on theoretical grounds, all British naval divers are now disqualified at the medical examinations from deep diving if they are inclined to fatness.

While a medical examination will probably exclude most of the

* *Journal of Hygiene*, 1908, p. 445.

Dr. Haldane. more susceptible individuals, it still seems very doubtful whether all can be excluded in this way. With new men, therefore, it is desirable that preliminary trials should be made with short shifts, as was the practice in the East River Tunnel work. These men should be carefully watched after decompression, and promptly recompressed if they show any threatening symptom; and any man who has shown threatening symptoms should be rejected.

With these precautions, in addition to having a medical air-lock and keeping the men within reach of recompression for about an hour after they come out, the writer thinks that in all probability Mr. Japp's plan would prove successful. For casual visitors, the duration of stay could be limited so that on coming out the maximum saturation in their bodies should not exceed 19 lb.

The conditions vary so widely in different kinds of work in compressed air that it seems hardly desirable to lay down by legislation or otherwise any hard-and-fast general rules as to decompression. The writer, however, thinks that for any particular undertaking in compressed air, special rules, suitable to the particular circumstances, should be drawn up and strictly enforced. If these rules could also have some authoritative sanction, as, for instance, in the case of "Special Rules" under the Coal Mines Regulation Act in England, engineers and contractors would be relieved to a large extent of what at present must be a very uncomfortable responsibility.

Dr. Keays. Dr. Frederick L. Keays* (by letter).—The prevention of compressed-air illness is, of course, most important. It is likely that with the modern demands of engineering, even greater compressed-air undertakings will be projected in the future than in the past. Any means, therefore, which will add to the safety of the workmen, will not only benefit humanity at large, but will also permit of more extensive undertakings.

The three great factors in the causation of compressed-air illness are: The degree of pressure, the time under pressure, and the decompression period. The degree of pressure is, of course, the chief factor; and it may be stated that, other factors remaining the same, the higher the pressure, the greater will be the dangers of compressed-air illness. The degree of pressure which is to be used in any undertaking is not a matter of choice, but depends on the depth to which the work must be carried. The only means of preventing illness, then, which are left to the engineer are the regulations of shifts and of decompression periods. In the past it has been thought that the longer one stays under pressure, the greater will be the dangers of compressed-air illness. This is true, up to a certain point, namely, that at which there is complete saturation of the blood and body fluids with the gases of the

* Late Medical Director for S. Pearson and Son.

air. Dr. Haldane has made a valuable contribution to the subject of compressed-air illness by his work on the question of body saturation, and Mr. Japp has worked out Dr. Haldane's theory in a plain and satisfactory manner. Dr. Keays.

The body fluids are practically saturated at any given pressure in about three hours. This belief is based on the results of the work in the Pennsylvania East River Tunnels. From a study of the records, it was found that men working for periods of less than 3 hours were less likely to have "bends," and much less likely to have a serious illness than those who worked for 3 hours or longer. Furthermore, it was found that in comparing the results of 6-hour shifts—which, since they were divided into two 3-hour periods with an interval of 3 hours at normal pressure, really meant 3 hours' exposure—with the results of 8-hour shifts, at almost equal pressures and with corresponding rates of decompression, there were fully as many cases of illness with the former shifts as with the latter, and certainly more serious and fatal cases. A point which has not been sufficiently emphasized in the past is that the real danger in compressed-air work comes with or after decompression. For this reason the practice, which has been almost universally followed, of dividing the working shift into two periods, is a bad one, for it doubles the number of decompressions. The results would be better in tunnel and caisson work, if continuous shifts of 4, 6, or 8 hours were arranged to suit the pressures; say, 4 hours for pressures of from 42 to 50 lb. +, 6 hours for pressures of from 32 to 42 lb. +, and 8 hours for pressures below 32 lb. +, and if the decompression periods were increased to reasonably safe lengths of time.

The question of the decompression period is a difficult one to settle. In the first place it must be admitted that certain men have individual susceptibility to the effects of compressed air. Just what the reason for this susceptibility is has never been learned. The fact remains, however, that while one man may work even at a high pressure and decompress rapidly without any symptoms of illness, another, under even more favorable conditions of pressure, length of shift, and time of decompression, may succumb to sudden death from syncope and collapse, and the autopsy will show large quantities of free gas in the circulatory system. Among practical compressed-air workers there is a decided difference of opinion as to the necessary length of decompression periods, some even claiming that quick decompression is preferable. Nearly all men, however, who have dealt with the subject from the physiological and medical standpoint, have advocated fairly long times for decompression. The writer believes that lengthening the time of decompression is the most efficient means of reducing the number of cases of compressed-air illness, as well as the number of serious cases.

On account of the common occurrence of cases of delayed onset

Dr. Keays. of symptoms, it is not believed that lengthening the decompression to periods claimed by Bert, von Schrötter, Hill, and Haldane, to be safe, would prevent all cases of illness, or even serious and fatal cases. In the writer's experience a comparatively large number of men first begin to have symptoms of compressed-air illness several hours after decompression, in some cases after as many as 12 or 15 hours, and in a fairly large number, after 3 hours. In two fatal cases symptoms began to appear between 2 and 3 hours after decompression. Why these men should not have had symptoms, even if they had taken as long as 1 hour for decompression, the writer cannot see. He believes, however, that reasonably long decompression periods are efficient in reducing the number of cases of illness, disability, and death.

Just what are reasonably safe lengths for decompression periods have never been determined. In the work on the Pennsylvania East River Tunnels after April, 1906, a rate of decompression of at least 1 min. for each 2-lb. pressure was required, and quite generally followed. This seemed to cause less illness and a reduction in the number of serious cases, as compared with previous months when there were no rules as to the rate of decompression. Even this rate of decompression, which was longer than has been required in any compressed-air work in the United States, did not prevent serious and fatal cases. For this reason the writer thinks that even slower rates of decompression should be tried, for pressures above 28 lb. +, below which, this rate of 2 lb. per min. appears to be safe.

Mr. Japp has set forth Dr. Haldane's method of "stage" decompression, which is claimed by the latter to be more efficient in preventing illness than uniform rates of decompression of even longer times. The writer does not think that enough experimental work has been done to prove the efficiency of "stage" decompression; but its use in diving has proved very satisfactory, and experience with the extra bulkhead in tunnel work, which necessitates an interrupted decompression roughly corresponding to Dr. Haldane's method, would argue favorably for "stage" decompression.

If it were possible, in accordance with Dr. Haldane's suggestion, to arrange a large decompression chamber in which a pressure equal to one-half the actual tunnel pressure could be maintained, where the men could pass a long decompression period in comfort, and use the time for bathing and dressing and, perhaps, for eating, it would probably prove a valuable means of preventing illness, and, at the same time, permit of working with comparative safety under higher pressures than could now be attempted.

Medical supervision and examinations are important and are instrumental in preventing serious results from the effects of compressed air. Men with organic disease are certainly less able to withstand the effects of compressed air than normal individuals. New men are

certainly more susceptible to the effects of compressed air than old workmen. When high pressures are being used, a preliminary test of about 1½ hours under tunnel pressure for all new men, followed by a second medical examination, and short shifts of from 1½ to 2 hours for the first working period, are effectual means of guarding against serious results among new workmen. These means proved very valuable in the work on the Pennsylvania East River Tunnels. Dr. Keays.

Much remains to be learned in regard to the prevention of compressed-air illness. This knowledge can be gained only by experimental work with animals, supplemented by records of compressed-air undertakings. By adopting new methods which appear to be improvements upon the old, we may hope to arrive, finally, at a more definite knowledge of what is reasonably safe in the lengths of shifts and decompression periods.

Compressed-air work in tunnels and caissons, where pressures above 30 lb. + are used, will always remain a dangerous occupation in spite of all reasonable precautions, for one can hardly hope to prevent all cases of illness. Those cases due to small quantities of free gas are almost sure to happen, and a small proportion of them is likely to result in paralysis, which may cause permanent disability or death. Until the reason why certain men are susceptible to the effects of compressed air, can be learned, and such individuals can be recognized, cases of sudden death from large accumulations of free gas are almost certain to occur occasionally.

Dr. Albert J. Loomis.*—The author deserves great praise for the clear and lucid manner in which he has stated the ideas of those who have investigated and written on the subject of compressed-air diseases. Dr. Loomis.

But after a pretty wide experience, extending back over twenty years or more, with pressures ranging from 15 to 42 lb., gauge pressure, the speaker cannot agree with him, or his authorities, as to the cause of the disease under discussion. While admitting the "saturation theory" as laid down by Bert, von Schrötter, Haldane, and others, the speaker does not think that it is the cause of the so-called caisson disease, for the following reasons:

First.—As to the formation of bubbles of gas or air in the blood-vessels themselves: They have only been demonstrated in exceedingly high pressures—15 or 20 atmospheres—which, at the present time, are considered impracticable as working pressures. If these bubbles should occur in the blood-vessels of the men, working as they do and coming out as they do, there would never be any use for hospital locks, as the men would all die of suffocation before they could be gotten to them, owing to these minute bubbles becoming entrapped in the very small

* Chief Medical Officer, North River Division, Pennsylvania Tunnels and Hudson Companies.

Dr. Loomis. vessels or capillaries of the lungs, and thereby preventing the oxygenation of the blood, a condition which operating surgeons frequently meet, to their great regret.

Second.—If the symptoms were due to any change in the blood itself, there would be a general set attacking all parts of the body to a greater or lesser extent, which is the very thing that does not happen, but, on the contrary, they show themselves in a very few localities, and most of these localities are in the lower extremities, situations which any surgeon who is at all proficient in locating nervous lesions by their symptoms, would place without hesitation in the spinal cord, and in the lower portion of the cord.

On the other hand, there is practically but one set of symptoms, the so-called "staggers," which would lead physicians to look for a cause or lesion in the brain, a condition which is quite rare when compared with the great number of cases occurring in the lower extremities.

Should there be any change in the composition or condition of the blood, the brain would naturally be the first organ to show such change, as it is without doubt the most delicate organ of the whole anatomy.

In reference to the CO_2, the engineers of the North River Division of the Pennsylvania Tunnels have prepared a chart on which all the cases of compressed-air sickness have been plotted, and this chart shows that, on that work, the greatest number of cases occurred at a time when the CO_2 was only 1 part in 1 000, with a pressure of from 20 to 25 lb.; and, likewise, when the pressure was 28 lb. and the CO_2 was 2 parts in 1 000, there were no cases of caisson disease.

While the speaker is not prepared to place much stress on the CO_2, other than as an indication of the general ventilation, he does think that weather changes play a very great part in the causation of the disease, as do also the habits of living of those engaged in the work.

When it comes to the examination of men, the medical officer should fit himself for the work by a close study of the conditions under which the men are to work, and his power of rejection should be absolute. The selection of the men should be the duty of one man who should be responsible for their welfare while they are at work under pressure.

In substantiation of these opinions, the speaker desires to state, that in the work of the Hudson Companies, in the construction of the Hudson and Manhattan Tunnels, about 38 000 men have been examined, since 1902, for pressures of from 15 to 42 lb., with three deaths, and in the construction of the North River tubes of the Pennsylvania Tunnels about 7 000 men have been examined for pressures of 15 to 38 lb., with no deaths.

Mr. Saunders. WILLIAM L. SAUNDERS, M. AM. SOC. C. E.—This is a subject about which very little of practical value has been written, and the speaker is glad to note that Mr. Japp has gone into the matter in so much detail, both from theoretical and practical standpoints.

Mr. Saunders.

Caisson disease, or the "bends" as it is commonly called, does not differ from other diseases, in that doctors are very apt to disagree about it. Heretofore the subject has been treated in print mainly from a medical standpoint, various theories being advanced as to the cause of this disease, that is, whether it is due to mechanical or chemical action. The speaker remembers when the most plausible theory advanced for the cause of caisson disease was that the blood, which had been compressed and held in a state of tension, was suddenly released by decompression, creating a pain when coursing through the veins. This was illustrated by the statement that if a string is wrapped tightly around one finger and then suddenly unwound, the finger is temporarily in pain. The subject has been discussed so fully by Mr. Japp, however, and the theory of caisson disease has been so well established by the experiments to which he refers that it is now generally accepted that bubbles are the cause of the "bends."

Compressed air is the speaker's hobby, and in his early experience in seeking information about it he has soared, like the eagle mentioned by Mr. Japp, to a height of 3½ miles, where he has felt the earmarks of the "bends," and has descended to great depths in mines and in a diving apparatus under the sea, where similar sensations have been felt.

Personal experience in a matter of this kind always fits one to judge better the causes as well as the effects, and it may add something to the interest in this matter to recite an experience which the speaker had some fourteen years ago in the old Hudson tunnel, now called the McAdoo tunnel. This tunnel was started by Mr. D. C. Haskin a little before, or about the time of, the Centennial Exposition in Philadelphia, in 1876. Mr. Haskin conceived the idea of driving a tunnel under the Hudson River by maintaining a chamber of compressed air to hold up the face. He used no shield, and as the material through which he worked was more or less unstable, he met with considerable difficulty and some lives were lost. He sank his entire fortune in the enterprise, the work being taken up later by English capitalists, who contracted with Messrs. Pearson and Son to finish the tunnel. At that time E. W. Moir, M. Am. Soc. C. E., was in charge of the work, and the experience which he gained there has no doubt helped him to design and carry through the recent tunnel construction under the East River, which Mr. Japp has so ably referred to and with which he has been connected in so important a position. During the administration of the Pearsons, the late R. P. Rothwell, M. Am. Soc. C. E., at that time editor of the *Engineering and Mining Journal*, and the speaker were invited to examine the tunnel for the purpose of making a report thereon. About an hour was spent under some 35 lb. absolute pressure. Coming out of the locks rather rapidly, on reaching the washroom, the speaker was taken with a severe attack

Mr. Saunders. of the "bends," the effect being like that of a paralysis of the lungs. A horizontal tank, resembling a common horizontal boiler, was at hand, and the patient was transferred to a bench of straw, the door was closed, compressed air was turned on, and the relief was instantaneous. A pressure valve, provided for the purpose of admitting and discharging pressure from the tank, was used to enable the prisoner to escape, but it was noticed that when the pressure fell only two or three points the "bends" returned, and it was only by the gradual expansion of the pressure, occupying about ½ hour, that the patient was able to open the door and walk out cured. The necessity for reducing the pressure gradually, as shown very graphically by the return of the "bends," indicates the value of the theory of gradual decompression. It also bears out the idea that time must be given to permit the escape of the little bubbles which accumulate in the body.

An interesting incident was noted in connection with this experience. When the pressure in the tank was lowered, through the discharge of air, a fog formed instantaneously, and as soon as air was let in and the pressure increased, this fog was as promptly dissipated. The speaker experimented with this for some time—manufacturing and annihilating fog. The lowering of pressure, of course, resulted in cooling the air, and, *vice versa,* increasing the pressure heated the air. As the air was cooled, fog was precipitated immediately when the temperature reached the dew point, the capacity of air for moisture depending entirely on volume and temperature, that is, a certain fixed volume of compressed air or free air can hold only a certain quantity of moisture at a certain temperature. If the temperature is raised, it will hold more water; if the temperature is lowered, the moisture must be precipitated first in fog, and later, in the form of rain.

Mr. Aims. WALTON I. AIMS, M. AM. SOC. C. E.—The literature on caisson disease is quite extensive. The causes of the disease now seem to be well determined, but difficulty is found in conforming practically with the medical rules and regulations for its prevention.

The age of the compressed-air worker has an important bearing on his ability to withstand the effects of too rapid decompression, but youth generally lacks experience, and, in difficult work, some compromise has to be made in drawing the age limit for entering the compression chamber. Dissipation acts strongly in undermining a man's resistance to the effects of compressed air, especially among workers of mature years, and even some temporary indisposition may occasion an attack in one who has previously been pronounced physically fit by a medical examiner.

In nearly all compressed-air work there is difficulty in compelling the workmen to take a reasonable time in the air-lock while decompressing. Believing that this is due largely to the extremely un-

comfortable conditions in the ordinary air-lock, a method for relieving the lock of the cold and fog when decompressing was used at the Battery-Brooklyn tunnel by introducing a constant flow of heated dry air while the lock was being exhausted. In addition to removing the cold and fog from the lock, this heated air also provided for ventilation while decompressing, a feature not found in the ordinary compressed-air lock. The comparative freedom from caisson disease at the Battery-Brooklyn tunnel, where practically no age limit was established, seemed to indicate the value of thus heating and ventilating the air-lock during the decompression period. Mr. Aims.

DR. SEWARD ERDMAN.*—The speaker wishes to acknowledge his appreciation of Mr. Japp's instructive contribution to the literature of caisson disease, and feels that the general adoption of the suggested "Step Decompression Method" will aid much in robbing compressed-air work of its gravest dangers. Dr. Erdman.

The reduction, to terms and curves of mathematical precision, of the time and pressure elements, as shown on his charts, is most interesting; but it seems to the speaker that there will always be another factor to be reckoned with, one which cannot be reduced to mathematical terms, and yet which will continue to render uncertain the result of exposure to compressed air in each individual case.

This indeterminate factor may be called the "personal equation," and seems to be dependent on the functional efficiency of the circulatory and respiratory apparatus.

In the experience of all who have seen much of compressed-air work, cases are not infrequent where men, who are seasoned air workers, will work daily, for long periods of time, at fixed conditions of pressure, length of shift, and decompression time, without suffering any ill effects; but who come to work the next day, under exactly the same conditions, only to develop a severe or even fatal attack of "bends."

The decompression time, indicated on Mr. Japp's charts, can be controlled by valves from the air locks; but, for the individual, desaturation of the body tissues and the blood must take place through the lungs; therefore the lungs act as the "escape valve" for the individual.

In explanation of such accidents as are above cited, it is likely that this "escape valve" is temporarily clogged.

Desaturation of any given part of the body will be impeded by causes which slow the blood current in that area.

Thus passive congestion of the lungs, or the presence of mucus in the air spaces (as from a cold) may act as clogs to the "escape valve," thus increasing the time necessary for desaturation, and, under such conditions, the body does not rid itself of the gases in the average time, and an attack of "bends" results.

* Formerly a Medical Officer on the East River Tunnels Contract.

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PAPERS AND DISCUSSIONS

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TESTS OF BUILT-UP STEEL AND WROUGHT-IRON COMPRESSION PIECES.

Discussion.*

By John C. Moses, M. Am. Soc. C. E.

Mr. Moses.

John C. Moses, M. Am. Soc. C. E. (by letter).—This investigation is to be welcomed as a forerunner of many more in which examinations of the actual conditions under working loads will be made. Tests to destruction, or to any point beyond the elastic limit, are of very doubtful value, and the practice of making them the basis for designing has been justified only by the supposed impracticability of making such tests as are described by the authors. Certainly nothing can be more illogical than to find that load, the first application of which will cause destruction, and then to assume that any arbitrary fraction of that load can be carried indefinitely with a proportional factor of safety. The absurdities of this method have become especially apparent when applied to such materials as reinforced concrete, but are none the less existent when applied to homogeneous materials like steel.

The difficulty of making the proper kind of tests does not alter the facts, and the proofs now presented that such tests are not impracticable are very encouraging. They will need to be many in number and to be made on structures when first erected and again on the same structures after they have been in use a while. They will need especially to be made on details of all kinds in order to eliminate the very large element of guesswork which now enters into this very important part of designing. Some of the authors' conclusions bear very significantly on this subject. They find "no relation * * * between the stresses actually observed and the stresses computed by

*This discussion (of the paper by Arthur N. Talbot, M. Am. Soc. C. E., and Herbert F. Moore, Esq., printed in *Proceedings* for April, 1909, but not presented before the Society, is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

column formulas," and suggest that the design of details and conditions of manufacture may be such as to make void the usual assumptions entering into these formulas. Mr. Moses.

The whole subject of detailing is connected intimately with that of manufacturing. It is of essential importance economically, but has not received its proper share of attention. At present, working stresses are commonly limited to one-half the elastic limit, principally because of the uncertainty which exists in regard to the proportion of the strength of the main members, which can be developed by the details. With more exact knowledge of the actual distribution of stresses, and with the advance in the art of designing details which will accompany this advance in knowledge, it may become apparent that higher cost of manufacture will be much more than offset by reduced expense for material. Better methods of manufacture will also make possible the use of stronger steels, with all the possibilities such a change foreshadows. The recent rapid increase in the variety of sections rolled shows that this study of detailing need not be confined to the sections which have been standard for the past thirty years.

The authors show by example that a bend of less than $\frac{1}{16}$ in. in the channels of Column No. 1 would account for an increase of stress of 50 per cent. Shop experience shows that, aside from kinks due to handling during shipment from the mills or about the shops, the actual punching of angles, plates, and channels will curve the separate pieces sufficiently to cause such eccentricities as that cited, although the parts are apparently straightened by the lattice. Channels will not only bend, but will also twist, when punched in the flanges. Their flanges will perhaps be kinked at every hole by the blow of the punch. Plates will show buckles between the rivets after driving. Twisters must often be applied to columns during riveting to keep them from assuming a spiral form. Designers insist that lattice bars of columns made of four I-shaped angles, shall intersect on a common rivet, and that the tie-plates shall be of the same thickness as a single bar, expecting the angles to be pulled down by the riveting. They are pulled down, with a resulting kink in the outstanding legs as well as in the riveted leg. Numerous examples of this kind could be cited in support of the authors' statement as to conditions of manufacture.

The authors' experiments on lattice bars show the desirability of more adequate provision for staying the open sides of columns, when such columns are used; and the frequent bending of lattice bars in shipment and erection emphasizes the argument to those acquainted with the facts. A return to closed sections with separate joint pieces is among the possibilities.

The accumulation of further data is to be earnestly desired, and will certainly have a marked effect on the future development of structural work.

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THE SEWER SYSTEM OF SAN FRANCISCO, AND A SOLUTION OF THE STORM-WATER FLOW PROBLEM.

Discussion.*

Sewers

BY MESSRS. W. C. HAMMATT, ROBERT G. DIECK, CHARLES E. GREGORY, E. KUICHLING, AND KENNETH ALLEN.

Mr. Hammatt.

W. C. HAMMATT, ASSOC. M. AM. SOC. C. E. (by letter).—In the writer's opinion, there are few cases when it is advisable to handle storm-water and sewage together; and it is particularly objectionable when the discharge is below water. For a given district, variations of the quantity of sewage proper are slight, while the storm-water will vary from nothing for a great part of the year to a quantity many times in excess of the sewage on some occasions.

In a main emptying below water level, therefore, and designed to carry both storm-water and sewage, the velocity would be so slight when sewage alone was being carried, that sedimentation would take place. Cases are conceivable where the sedimentation would be so great during prolonged dry seasons that the mass would be too great for the first storm to remove, and serious damage might be done to the outfall piping and the lower sewer districts.

It might be claimed that the relief outlets would take care of all the storm-water if the main pipe were obstructed in this way, but if such a thing should happen, and the storm were of a low intensity, the sewage which would take the relief outlet would be diluted so slightly as to be offensive, and as the relief sewers are to empty in exposed places, this would be objectionable.

*This discussion (of the paper by C. E. Grunsky, M. Am. Soc. C. E., printed in *Proceedings* for March, 1909, and presented at the meeting of May 19th, 1909), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

Mr. Hammatt.

The writer believes that sewage proper and storm-water should be handled in separate conduits and under separate systems. The sewage should be discharged below low-water level in such a manner as to produce proper dilution and removal by currents, as described by Mr. Grunsky. The storm-water should be discharged above tide level, when possible, at the most convenient beach or pier head. The outfall for sewage proper should be of such cross-section that the velocity against the back-water would never fall below 0.5 ft. per sec.

Mr. Dieck.

ROBERT G. DIECK, ASSOC. M. AM. SOC. C. E. (by letter).—In connection with the design of an improved sewerage system for San Francisco, it may be of some interest to state the method of design followed in the storm-water drainage system of Manila, P. I. To understand clearly the essential geographical features of the city a few remarks are necessary. The city is situated at the mouth of the Pasig River, on the east shore of Manila Bay, on an alluvial plain formed of sand and gravel, mingled with loam, brought down from the range of mountains which forms the main drainage slopes of the Island of Luzon. In addition to the major stream, the river has two main branches which take off within the limits of the city, and serve as additional reliefs in times of high tide and floods in the river. One branch extends north and the other south, and both are of about equal importance. There are several smaller streams, which intersect these branches and the main stream, forming a network of canals of more or less importance, which makes the system of waterways practically interdependent. There are also several minor connections to the bay, but these are of lesser importance. Except at extreme low water, practically all the streams are navigable for small river boats, and furnish a ready means of transportation. The larger streams have been dredged where shoaling has occurred.

The ground surface rises from about 2 m. above mean low water in the southern and western portions of the city to about 5 m. above the same datum in the northeastern portion, where small foot-hills are encountered. Practically 75% of the area of the city lies within these elevations. As the average range of the tide does not exceed about 1 m., water is encountered in portions of the town at a depth of 1 m. below the street surface at high tide. Because of this low elevation, the water-table is almost constantly near the surface in large areas of the town; and, for the most part, in the business and residence sections, trenches more than 1.5 m. deep require to be pumped. As these sections required attention at once, it was soon seen that the shallow trenches made necessary by the high water-table would influence seriously the design of the sewers. During the rainy season, which occurs from about June to late in October or even to early December, as a consequence of the torrential rains, the water-table rises, and, in some parts, almost coincides with the street surface, making dry trenching an impossibility without pumping.

Mr. Dieck.

The river has a depth of about 14 ft. at the east end of the harbor (obtained by dredging), decreasing to about 6 ft. at extreme low water at the eastern limits of the city. The main branches have a maximum depth of about 8 ft. Gaugings of these streams and the river have not been made, but, in times of flood, when the current has a velocity of nearly 10 ft. per sec., the discharge of the river must be at least from 50 000 to 60 000 sec-ft. For the most part, the *esteros* (as the branches are called) are sluggish and rather shallow, exposing large portions of their beds at low water. Their frequency, however, makes them admirable recipients of storm-water, and makes further subdivision into drainage districts unnecessary.

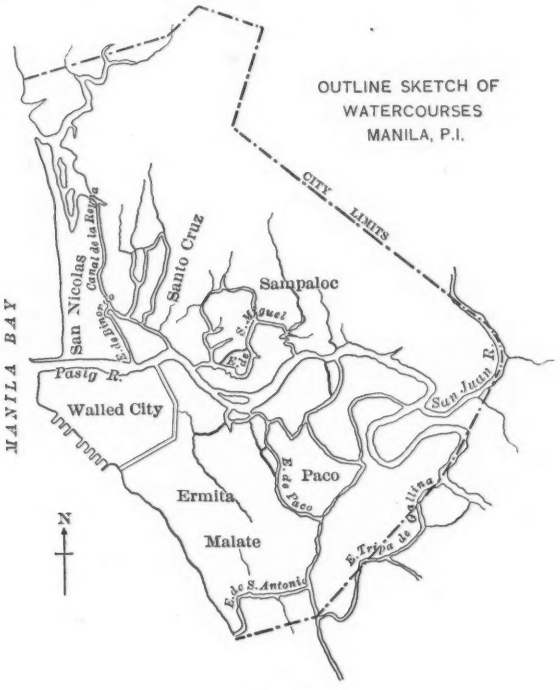


FIG. 12.

Before it was possible to arrive at a satisfactory solution, it was evident that the entire question of street grades would have to be considered, and that some definite plan for these, in opposition to the former, haphazard plan, would have to be adopted. To this end, careful levels were taken throughout the city, and the data were marked on the city plans. Preliminary computations clearly showed that to avoid excessive fill, which, under local conditions, would have proven extremely costly, and to protect property interests against serious

damage following violent changes in grade, a flat gutter grade was absolutely essential. After long discussion, this grade was fixed at the absolute minimum of 0.002. It had been observed that water would stand, on the most carefully constructed work laid to a grade less than this, and as all stagnant water is objectionable in the tropics, it was held that this feature should govern the work, even if in street construction the item of filling were large. As a further limiting factor, it was fixed arbitrarily that the street surface at no point should be less than 0.75 m. above the highest tide. The limits set varied for the various districts. Although it was known that all the low districts would be subject to overflow during high floods in the river, it was still held that some reasonable height of street surface should be fixed in all cases, in order to improve, wherever possible, a naturally bad condition. Mr. Dieck.

Another limitation was in reference to cover. After some observation of 24-in. concrete pipes, under heaviest city traffic, the minimum cover was fixed at 0.60 m. Up to the present this cover has been sufficient. For reinforced sewers no limit was set except an amount sufficient to clear the paving. Under the worst conditions, therefore, with the further provision that the sewers should not be laid below mean low water, because of the likelihood of silting, the sewers had to be designed to fall entirely within from 1.15 to 1.25 m. It was essential, also, that the space occupied by the sewers should be kept at the lowest limit in order to provide for other structures in the narrow streets, these, in some cases, being only 8 m. between building lines, with a clear roadway of only 6 m. At every turn, therefore, certain definite restrictions were to be observed, and also the strictest economy in space and head-room.

A study of the rainfall records was then made. Fortunately, the admirable records of the Philippine Weather Bureau were available, and were examined for maximum intensities for a period of at least 16 years. The records clearly showed that intensities of more than 3.00 in. per hour were not uncommon. Table 20 is a list of typical storms which were considered as probably the worst to be encountered.

Storms having an intensity of 2 in. were common throughout the rainy season, and occurred when the ground was thoroughly saturated, as well as during the drier times. It was evident, therefore, that no relief could be expected from ground storage, but, as the runs in the sewers were short, it was decided that the maximum flow could not occur until about 10 min. after a severe rain had commenced. Table 20 shows that ordinarily the very heavy rainfalls did not last more than from 20 to 25 min., so that if the maximum discharge occurs as stated, and provision is made for lesser rainfalls, there can be flooding only for 10 or 15 min. As there are no cellars or basements in the city, provision against back-water in the sewers was not required, and

Mr. Dieck. it was considered that with safety and at the expense of only a temporary inconvenience, flooding of the roadways would be permissible during unusual storms. Under such cases the sewers would run under a head, which would increase their capacity.

TABLE 20.—TYPICAL STORMS IN THE PHILIPPINE ISLANDS.

Date.	Total rainfall, in inches.	Duration, in minutes.	Excessive rainfall, in inches.	Hourly intensity, in inches per hour.
Apr. 29, 1905	2.56	14	0.79	3.50
May 21, 1892	2.36	30	2.36	4.72
June 8, 1905	2.26	10	0.98	5.90
" 15, 1891	2.16	8	0.59	4.43
Sept 18, 1887	2.06	30	1.57	3.05
Aug. 26, 1903	2.05	30	1.62	3.24
July 19, 1899	2.02	15	Record imperfect.	
Sept. 15, 1891	1.98	12	0.785	3.94
July 16, 1890	1.98		Record imperfect.	
Nov. 16, 1891	1.96		Record imperfect.	
June 1, 1902	1.91		Record imperfect.	
Aug. 6, 1889	1.89	7	0.59	5.51
June 30, 1889	1.86	20	1.18	3.54
Sept. 14, 1898	1.86	17	0.985	3.48
Aug. 27, 1888	1.85		Record imperfect.	
Sept. 17, 1888	1.85	35	1.79	3.07
July 13, 1904	1.81	10	0.73	4.37
Aug. 28, 1897	1.81	16	1.57	5.90
Mar. 9, 1894	1.80		Record imperfect.	
July 12, 1904	1.75	10	0.79	4.72
Sept. 6, 1896	1.73	15	0.985	3.15
July 23, 1888	1.68		Record imperfect.	
Aug. 26, 1886	1.65	15	0.59	2.36
May 29, 1891	1.65	10	0.787	4.72
Sept. 20, 1904	1.62	10	0.71	4.25

It was then decided to design the sewers under the assumption of a 2-in. rainfall, discarding the excessive storms from consideration, relying on the sewers under pressure to discharge the excess, and permitting the flooding of the roadways for 15 min. In the design no account was to be taken of the increased capacity of the sewers due to this head, the excess being viewed merely as a factor of safety in the design. This compromise was made only after the closest preliminary trials and after it was clear that the restricted head-room, flat grades, and the necessary economies in expenditures required some concessions to be made. It was also kept in mind that during severe storms the river and the *esteros* were subject to overflow and that this could not be prevented by any possible provision in the sewers. The results obtained under this assumption, while not satisfactory in all cases, appear to be justified, for no serious flooding had occurred in 1907, two years after the first sewer was built under the general plan.

At first, a run in the gutters of 120 m. had been fixed as the maximum in the built-up sections, but later observations on the gutter flow showed that this was excessive, and the maximum was reduced to 100 m. Sewers with inlets at this distance apart give better satisfac-

tion, and flooding has been lessened. All inlets have direct connections to the sewers, and are provided with silt basins and grating covers. Mr. Dieck.

In the designs, Kutter's formula for discharge, with sewers running full, was used, the values of n being taken as follows:

0.011 for cement plaster finish,
0.013 for clean brick and glazed pipe,
0.015 for local stone.

A velocity of 3 ft. per sec. was sought wherever possible, so as to lessen the likelihood of silting, which had always been the greatest difficulty in the old sewers built under Spanish direction. All drainage areas were kept completely separated, and the connection between the old sewers built by the Spaniards was stopped in order to make the direction of flow definite. To a great extent, the deposits were undoubtedly due to this error in design, and the immediate betterment in the condition of the old sewers following the removal of the connections was made evident by the increased discharge. Wherever possible the old Spanish sewers were utilized, the sides and inverts being made smoother by cement plaster.

In the absence of any data on the character of the run-off, an assumption of 75% was at first made; but this was found to be excessive in large sections and deficient in the closely built-up parts of the Walled City and in the business sections. The final run-off coefficients were fixed as follows:

Heavily built-up	0.90
Ordinary business and residential	0.75
Suburban	0.50

Undoubtedly, these high coefficients lessened the flooding of the streets and are ample for all future work. It is difficult to compare the assumptions with the results obtained by Emil Kuichling, M. Am. Soc. C. E., or with those by Mr. Grunsky, but it is clear that there has been no serious error in the designs.

The *esteros* are in need of improvement, but their betterment would involve the city in such enormous expenditure that no definite plans to this end have yet been evolved. A large amount of dredging is necessary, and the streams must be confined to definite channels by walls. A preliminary section for a light shallow wall of reinforced concrete was designed, but the preliminary estimates of cost were so appalling that this could not be recommended. These *esteros* are the natural reliefs of the river, and their improvement is of the highest importance in the scheme of storm-water drainage. At times the velocity of the current does not exceed 1.0 ft. per sec. and no scour of the bed is possible. A deepening of the streams would result in increased flow, and their present unsanitary condition would be removed. It is quite probable that there must be some control of their

Mr. Dieck. currents, by gates or otherwise, if the reduction of deposits is to be secured, but, at present, there seems to be no necessity to consider that in the design of the confining walls.

From these remarks it will be clear that the drainage problem of Manila has many nice points to recommend it to the attention of hydraulic engineers, and while no claim is made to any originality, it is believed that the solution is somewhat out of the ordinary.

Mr. Gregory. CHARLES E. GREGORY, ASSOC. M. AM. SOC. C. E.—This elaborate paper presents an unusually able theoretical solution of the problem of run-off in storm-water sewers, and is most valuable as an academic discussion of the subject. In the speaker's judgment, however, the author has not given enough weight to the influence of storage on the surface of roofs and streets. After making certain assumptions for a hypothetical case, this part of the problem is very ingenuously and correctly solved for that case. The assumption is made that the rain reaches the sewer from the roofs and streets in maximum volume within 5 min., which is similar to the assumption often made by supposedly careful investigators and writers. The author, however, shows a much better appreciation of what actually occurs than they in that he takes account of the storage on the surface for the time assumed. For many roofs and a few street surfaces this assumption of 5 min. is no doubt approximately true, but a careful consideration of what should occur, theoretically, and a few observations of what actually does occur, will indicate a wide variation from the assumed 5 min., and demonstrate that storage on the surface, particularly for short, intense storms, has a much greater influence on the run-off than is indicated in the paper.

It is not the speaker's purpose to attempt to solve this very interesting and intricate problem completely, but simply to present a few studies of storage on catchment areas, which have been made for the solution of special problems, and to show their relation to the general subject.

Referring to a paper by the speaker and the discussion thereon by A. Marston, M. Am. Soc. C. E.,* it was there brought out that:

"*First.*—The water falling on the so-called 'impervious' area of an ordinary sewer water-shed does not all run off as fast as it falls, but part of it accumulates in increasing quantity on the surface during downpours of moderate length, such as cause the maximum discharges from sewer districts of ordinary size.

"*Second.*—As the storm continues at the same rate, the ratio of run-off from the 'impervious' area to the rate of rainfall increases, owing to the increased depth and velocity of the surface flow toward the sewer, until finally, if the storm lasts long enough, the rate of run-off from the 'impervious' areas becomes equal to 100% of the rate of rainfall."

* "Rainfall, and Run-off in Storm-Water Sewers," *Transactions*, Am. Soc. C. E., Vol. LVIII, p. 498.

Rough observations indicate that the depth of water which accumulates during heavy downpours on an ordinary street, sidewalk or roof, is greater than is generally assumed, and that the mean velocity of flow in the gutters is not as great as it is usually computed to be. Mr. Gregory.

The accumulation of rain on a street has been considered theoretically in the following manner:

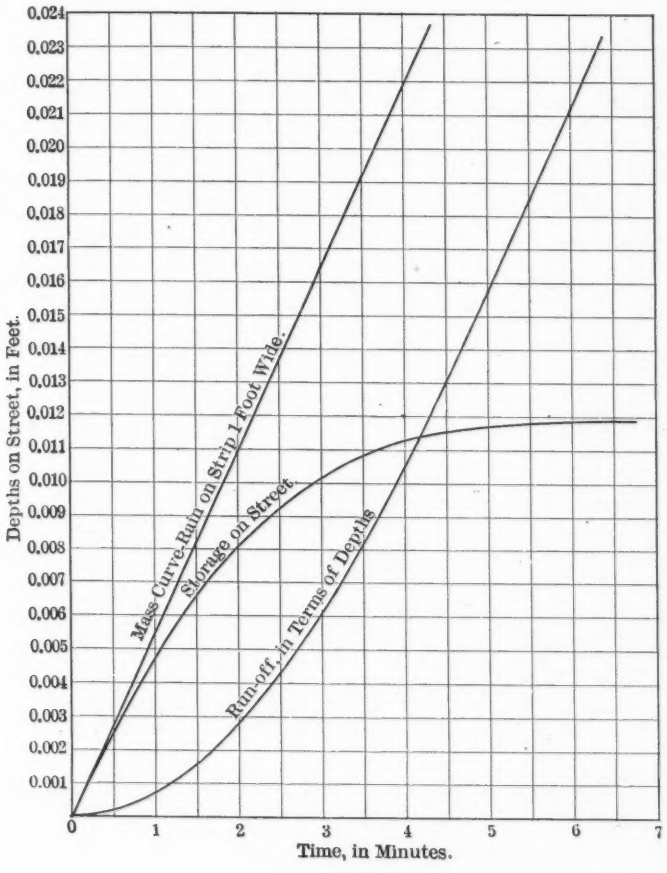


FIG. 13.

The laws and coefficients of flow are not very well determined for very shallow depths, and the impact of the falling rain and other factors materially retard this flow. It was assumed, however, that Kutter's formula was applicable, and, to allow for these retardations, a high value of n was assumed. As an asphalt street, from which the greatest flow may be expected, has a low crown, it was assumed, as

Mr. Gregory.

has been observed to be roughly true, that the accumulation of depth would be about uniform over its entire surface. From these assumptions, the diagrams on Fig. 13 were deduced. Three curves are there shown. The mass curve of depths for the entire rainfall is for a rate of 4 in. per hour. Points for the other two curves were obtained from the expression $t = \frac{S(H_2 - H_1)}{I - \frac{Q_2 + Q_1}{2}}$, by plotting values of H_2 and Q_2 in terms of depths.

t = increment of time, in seconds, for surface to rise from H_1 to H_2;
S = area of unit length of half the street = 12 sq. ft.;
H_2 = depth at end of increment of time, t;
H_1 = depth at beginning of increment of time, t;
I = rate of rain, in cubic feet per second, falling on the area, S;
Q_2 = flow, in cubic feet per second, at edge of gutter, due to depth, H_2;
Q_1 = corresponding flow at beginning of time, t, for depth, H_1.

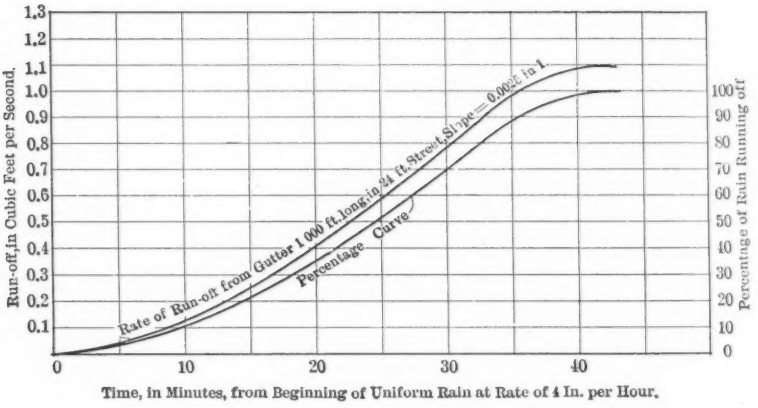


FIG. 14.

As an extreme example of storage in streets, the following is presented:

The run-off from, and the storage in, the gutter of a 24-ft. street, 1 000 ft. long, with a slope of 0.0025 in 1, is computed in the same general manner as for the street surface. The run-off curve is shown on Fig. 14. For a gutter, or a surface with a uniform slope, the formula is modified to take account of the varying depths, as follows:

$$t = \frac{S(H_2 - H_1) - (A_2 V_2 T_2 - A_1 V_1 T_1)}{I - \frac{(Q_2 + Q_1)}{2}}$$

In this case, t_1, H_2, H_1, Q_2, and Q_1 have the same meaning as before, but are applied to flow in the gutter. Mr. Gregory.

S = average width of each increment of depth multiplied by 1 000 ft.;

I = 1 000 times the inflow from each unit of length, and is taken from the curve of run-off, Fig. 13;

T_2 = total times elapsed from beginning of uniform rain in seconds, or until the depth, H_2, is reached;

A_2 = difference in area between the wet cross-section of gutter for the depth, H_2, and the average wet cross-section for the upper reach of the gutter where the depths are variable;

V_2 = average mean velocity in the gutter until the depth, H_2, is reached;

T_1, A_1, and V_1 bear the same relation to H_1 as T_2, A_2, and V_2 bear to H_2.

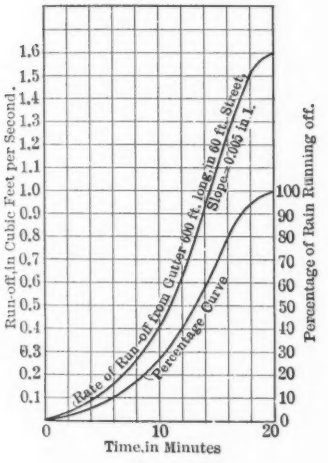


FIG. 15.

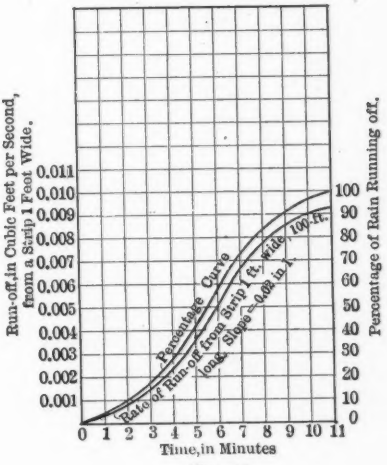


FIG. 16.

On Figs. 14, 15, and 16, the rate of the run-off curves is deduced by the same general formula for a rain at the rate of 4 in. per hour, but the storage and inflow curves are omitted, and a curve showing the percentage of rainfall running off is added.

Fig. 15 is for a street 60 ft. wide and 600 ft. long, with an asphalt pavement and a slope of 0.005 in 1.

Fig. 16 is for a roof 25 ft. by 100 ft., sloping at the rate of 0.02 in 1 toward a gutter across the 25-ft. side.

There are a great many roofs and streets comparable to those assumed for the curves on Figs. 15 and 16. A steeper slope, if paved

Mr. Gregory. with stone blocks, would give results equivalent, for flow and storage, to the flatter smooth surface. The Sixth Avenue Sewer District gaugings have been reported by the speaker.* This district contains many 600- and 900-ft. blocks, also many flat roofs, 25 ft. by 100 ft., or larger. It is estimated that it takes 15 min. for water to flow through the sewers from the remote parts of this water-shed to the gauge. In a sewer, the rate of run-off from roofs and streets, as shown in Figs. 15 and 16, may be estimated by assuming that this catchment area is divided into five concentric zones, the outside limits of each being 3 min., 6 min., 9 min., 12 min., and 15 min., respectively, from the gauge. The area of those zones may be taken from the gauge-point outward in the ratio of 1, 2, 3, 4, and 5. The average rate of run-off, expressed as a percentage of the rainfall, for the entire area at the end of 15 min. of uniform rain, at the rate of 4 in. per hour, is found by averaging the percentage of run-off for the average time length of each zone read from the percentage curve on Figs. 15 and 16, respectively.

The averages for the street and roof areas are thus found to be 11% and 48%, respectively. If the catchment area was of such shape that all the zones had equal areas, the percentages would be about 20 and 63, respectively. In a similar manner, a 5-min. time length gives a run-off of about 70% of the rainfall. It is not likely that a rate of 4 in. per hour would be maintained for more than 10 min., and the run-off from the nearby areas would be greatly reduced for the last 5 min. The actual gauged run-off from this area was about 40% for a rain lasting 15 min. This observed percentage may be accounted for by assuming the area to be made up of roofs and streets varying from 5 to 20 min. in time length, taking into account the storage in the catch-basins and sewers, and making allowance for pre-maximum and post-maximum rain rates, the greater influence being exerted by the surface storage. For such rates of rainfall as occur in San Francisco, the velocities of the surface films, due to less depth, would be much less than for a 4-in. rate, and, consequently, the time lengths for similar areas would be greater and have correspondingly greater influence on the run-off. The speaker wishes particularly at this time to call attention to the great influence exerted by the shape, size, character, and slope of each area, and by the frequency of inlets from the streets.

Mr. Kuichling. E. KUICHLING, M. AM. SOC. C. E.—The author deserves the highest credit for having made a more complete analysis of a complicated hydraulic problem than has heretofore been published. The novelty in his procedure is the estimation and consideration of two factors, the first being the quantity of storm-water temporarily in transit on the surface toward the various sewer inlets; while the second is the quantity temporarily stored in the sewers themselves. Both these

* *Transactions*, Am. Soc. C. E., Vol. LVIII.

factors tend to reduce the rate of maximum discharge by making the outflow somewhat less than the inflow until the conduits become entirely filled. Mr. Kuichling.

The first factor is obviously dependent on the extent and slope of the territory tributary to each sewer inlet or catch-basin. If the entire area were impervious and divided into a number of equal or equivalent hopper-shaped parts, each having a catch-basin in its middle point, and in which the time required for the water to flow at uniform velocity from the boundary line to the central basin is t_o minutes, then the quantity of storm-water which will still be on the surface at the end of such time is shown by the author to be two-thirds of the precipitation on the total area during that time.

It should be noted, however, that uniform velocity of flow over the surface of a hopper-shaped area involves a constantly reducing slope for the successive elementary zones, as the depth of the water is constantly increasing from the periphery to the central catch-basin. Such a condition of surface is rarely encountered in municipalities, and hence it must be assumed that the velocity increases steadily as the water passes successively from the larger exterior elementary zones to the smaller interior ones on the same grade. In general, the velocity in a channel, or on a surface, is proportional to the square root of the depth; and as the depth increases very rapidly on the successively reducing elementary zones of like grade, it follows that much less than two-thirds of the precipitation on a uniformly graded component area will still be on the surface in transit to the catch-basin at the end of t_o minutes.

To illustrate this feature of the case by an example, assume a hopper-shaped area, 1 000 ft. square, having each of the four equal sides graded on a uniform slope to the central catch-basin. Conceive the area divided into five zones, each 100 ft. wide. Beginning at the basin the areas of these zones will be 40 000, 120 000, 200 000, 280 000 and 360 000 sq. ft., respectively, or in the proportion of 1, 3, 5, 7, and 9. Next assume that the rain falls at uniform intensity for a sufficient length of time to allow the run-off from the outermost zone to reach the basin. Now, if it were possible for the velocity to be uniform, as well as the slope, the water flowing over the innermost zone would be about 25 times as deep as on the outermost one. With a uniform grade, however, the velocity will increase from zone to zone, and the depth in the vicinity of the basin will be materially less than the amount just mentioned, since the time of transit is greatly reduced. If the velocity in each zone varies with the square root of the depth of the flow, the time of transit from boundary to center will be less than three times as long as that across the outermost zone, and the quantity of water remaining on the surface at the end of such time will be considerably less than two-thirds of the precipitation during that time.

Mr. Kuichling.

In urban districts, the catch-basins or sewer inlets are generally much less than 1 000 ft. apart, on the average, and the time required for the run-off from the boundaries of the tributary areas to reach the inlets is usually not more than 10 min., as the water soon reaches the gutters, in which the flow is much more rapid than on a plane surface. During this period of time, the precipitation may be 0.5 in. in depth over the entire territory, and perhaps one-half the water may remain on the surface in transit to the inlets, but, at its termination, the surface storage comes to an end if the rain continues as before, and the run-off will thereafter proceed at its full rate to each inlet, since the storage on the surface is not cumulative when the intensity of the rainfall is uniform. For large districts, in which the time of passage through the longest line of sewers is 30 min. or more, during which time a precipitation of 1.5 in. may occur, the retention of a depth of 0.25 in. on the surface during the first 10 min. of the downpour is not of much significance.

The same may also be said with respect to the storage in the network of sewers, as the cubical contents thereof do not often exceed an average rate of 1 000 cu. ft. per acre, including house connections. This capacity is equivalent to a depth of 0.28 in. of rainfall; but, as the whole of the precipitation does not reach the sewers, owing to various causes of loss, it may be assumed that the capacity is equivalent to the run-off from a precipitation of about 0.40 in. In heavy rainfalls of comparatively long duration, however, this storage capacity will usually become filled some time before the end of the storm, and surcharge of the sewers will ensue unless their discharging capacity is adequate.

There can be no doubt as to the propriety of taking these two factors of surface storage and conduit storage into consideration for small districts and short rainfalls, as the aggregate effect may often be very appreciable; but, in the case of large urban districts, having times of transit through the longest line of sewers of more than 30 min., it seems to the speaker that the aforesaid effect will be lost in great degree.

The essential factor in the problem is the relation of the intensity of the precipitation to its duration. Usually, this is estimated more or less arbitrarily from inadequate data, and without an exact knowledge of the extent of territory covered by the rainfall at the recorded intensity. In most cases, also, the extreme observations are disregarded, and the rainfall curve is adapted to the more frequent downpours. If the sewers are proportioned closely to such a curve, there will be no margin of capacity, and it may be questioned whether a moderate reserve capacity, such as would be gained by leaving the above-mentioned two factors out of account, is not a desirable feature. It would certainly be interesting if the author would indicate quantita-

tively how the full consideration of the said two factors would affect the design of a large sewer system in the Eastern States. Mr. Kuichling.

In regard to the rainfall curve that should be used in designing sewers, it has long seemed to the speaker that only those durations should be taken into account for which the intensity or rate per hour was uniform, or nearly so. The object is to ascertain the probable maximum run-off from districts of different magnitude, or duration of flow through the longest line of sewer. Now, it is obvious that the run-off from a homogeneous surface will become largest when the rainfall is greatest and is likewise distributed uniformly over the entire area for a sufficient length of time to allow the water from the outermost boundary to reach the point of observation at the foot of the territory; hence it is of primary importance to deduce from the rainfall data the maximum uniform intensities for different periods of time.

The next step is to determine whether this relation is limited to areas of any particular magnitude, or whether it is possible to obtain a larger rate of run-off at the foot of an extensive area by using precipitations of varying intensity, but having an aggregate duration equal to the time of flow through the longest line of sewer. In general, it will be found that, if the relation between the maximum uniform intensity and its duration has been established correctly, the rate of run-off at the foot of a homogeneous territory of any size will be greatest when the entire area is covered by a rainfall of the highest uniform intensity corresponding to the length of time required for the water to flow to the point of observation from the outermost boundary. Deviations from this rule, however, may possibly occur when the area is not homogeneous, and in such cases one must be governed by the given or assumed conditions of the particular problem.

An instance of non-homogeneous territory is afforded by the following example: Let A equal the number of acres at the foot of a district, with a run-off factor equal to f_1, and the time of flow through the longest line of sewer equal to t_1 minutes; B, f_2 and t_2, C, f_3 and t_3, etc., equal the same elements for the next succeeding component areas; also, let the maximum uniform intensity of the rainfall, in inches per hour (or cubic feet per second per acre), be expressed by $I = \frac{a}{t + b}$, where t is the duration, in minutes. The rates of run-off, in cubic feet per second, from the several component areas, at the end of the time $T = t_1 + t_2 + t_3 + \ldots$ will then be $q_1 = f_1 A I$, $q_2 = f_2 B I$, $q_3 = f_3 C I$, etc., and their sum at the foot of the district will be:

$$Q = (q_1 + q_2 + q_3 + \ldots) = I (f_1 A + f_2 B + f_3 C + \ldots) = \frac{a}{T + b} (f_1 A + f_2 B + f_3 C + \ldots).$$

Mr. Kuichling.

If the district were entirely homogeneous, we would have $f_1 = f_2 = f_3 = \ldots = f$, and hence, also:

$$Q^1 = I f (A + B + C + \ldots) = \frac{a f}{T + b} (A + B + C + \ldots) = \frac{a f N}{T + b},$$

where N denotes the total area of the district $= (A + B + C + \ldots)$.

It will be noticed that if the run-off factor, f, represents the true mean for the whole district, the values of Q and Q^1 will be equal, thus indicating that it is immaterial whether the surface is homogeneous or not.

To pursue the subject a little further, conceive the territory to be homogeneous and divided into equal squares along the sides of which the run-off from each square, or aggregation of squares, moves with equal velocity, v_0, in feet per second; also let the two consecutive sides of each square have an aggregate length of 60 v_0, corresponding to the path traversed by the run-off in 1 min. At the end of T minutes, the number of squares contributing to the discharge at the lower corner of the first square will be $T \frac{(T+1)}{2}$; and, as the area of each square is $m = (30\ v_0)^2$ square feet, the total tributary area, in acres, will be

$$N = \frac{T(T+1)\,m}{2.43560}, \text{ with } Q = I f N = \frac{a f m}{87\,120} \times \frac{T(T+1)}{T+b} = c \times \frac{T(T+1)}{T+b}.$$

We thus have Q as a simple function of T, the form of which shows that Q increases directly with T and attains its greatest value only when T is greatest, or when the entire area contributes to the discharge. This result is due to the peculiar form of the expression for the factor, I. With some other form for I it may happen that Q will become a maximum for a value of T less than that corresponding to the time for flowing through the longest line of sewer in the district.

From this brief analysis of the run-off problem, it appears that attention should be directed mainly to the proper form of expressing the maximum uniform intensity of the rainfall, I, in terms of its duration, t, in minutes. By the use of this factor, the indefinite question of seeking a maximum, Q, with rainfalls of variable intensity during the total time, $T = (t_1 + t_2 + t_3 + \ldots)$, will be entirely avoided, as the former necessarily embraces all combinations of the latter.

The next matter of importance is the investigation of the proper value of the run-off factor, f; but, as the author has not given the results of his own experiments, there is no reason for discussing it here at considerable length. Few direct and reliable measurements of the run-off from a given territory and uniformly distributed rainfall appear to be available. In some cases the rainfall is observed at only a single point in or near the district, and no proof of uniform distribution is offered, while, in other cases, both mean intensity and

distribution are estimated. It is very desirable, therefore, to increase the number of accurate measurements for surfaces of different character, and to include therein the actual time of passage through the longest line of sewer, as well as the time required for the water flow over the surface from the outermost boundary to the nearest catch-basin or inlet. Mr. Kuichling.

To exhibit the range in the estimated values of the run-off factor, f, or the ratio of the run-off to the rainfall at the time of maximum discharge, the following data are submitted:

For roof surfaces assumed to be water-tight...	$f =$	0.70 to 0.95
" asphalt pavements in good order.....		0.85 to 0.90
" stone, brick, and wooden block pavements with tightly cemented joints.....		0.75 to 0.85
" same with open or uncemented joints....		0.50 to 0.70
" inferior block pavements with uncemented joints		0.40 to 0.50
" macadamized roadways.....		0.25 to 0.60
" gravel roadways and walks.....		0.15 to 0.30
" unpaved surfaces, railroad yards and vacant lots.....		0.10 to 0.30
" parks, gardens, lawns, and meadows, depending on surface slope and character of subsoil.....		0.05 to 0.25
" wooded areas or forest land, depending on surface slope and character of subsoil..		0.01 to 0.20

Others do not attempt to make close estimates of the different kinds of surface in an urban district, but content themselves with general average values of f, as follows:

For the most densely built-up portion.....	$f =$	0.70 to 0.90
" the adjoining well built-up portion.....		0.50 to 0.70
" the next adjoining residential portions, with detached houses and buildings.....		0.25 to 0.50
" the suburban portions, with few buildings.		0.10 to 0.25

Similar variations are found in the estimates of maximum intensity of rainfall for which provision is to be made in the sewers or other drainage channels, even in neighboring localities where the rainfall has the same character and average annual depth. In view of such large differences in practice in the choice of f and I, there is little encouragement for the adoption of refinements in the general theory of the subject.

KENNETH ALLEN, M. AM. SOC. C. E.—It is a good thing to place on record the conditions governing the design, and a description of the essential features, of an important improvement of this kind. Mr. Allen.

Of the fifteen cities of the United States having in 1906 a popula-

Mr. Allen. tion of more than 300 000, all are situated near bodies of water of such size that up to the present their sewage is discharged into them in a crude condition. Baltimore, however, in order to protect the oysters of Chesapeake Bay, is constructing a purification plant, and in order to avoid an undue pollution of the water, Chicago, San Francisco, Boston, and Washington have spent large sums to secure more favorable points of outfall. Without question, in view of increasing populations and a growing sensibility to cleanliness and sanitation, the next two decades will see extensive improvements in most of the other cities in the list.

Of these fifteen cities, four are on the Great Lakes, five (including Chicago) drain to the Mississippi or Ohio Rivers, and six deliver their sewage almost directly to the ocean. San Francisco, however, is especially favored in being able to locate her outlets—ultimately, at least—in strong currents of deep water which trend off shore and soon pass out to sea. New York and Boston are also located on great tidal bays, but the waters of Upper New York Bay oscillate between the Hudson and East Rivers and the Narrows in close proximity to a dense population for several days before being finally dispersed in the Lower Bay or ocean, while Massachusetts Bay is studded with islands, and the shores are lined with towns and resorts, so that great care has been required in selecting points of outfall to avoid offense.

With a discharge into large bodies of water there should be sufficient dilution to prevent putrefactive conditions, and the problem resolves itself into one of rapid dispersion. This can be secured with a single outlet where the current is swift in comparison with the volume of sewage. At San Francisco, as at Boston, interception and a discharge at a few well selected points, after the removal of the grit and coarser floating matters, has been decided on as the best plan. To secure a more rapid dispersion, multiple outlets are provided in the Elbe at Hamburg, and have been proposed for the Passaic Valley Sewer in New York Harbor; but where cities are located on sluggish streams, it is sometimes preferable to maintain numerous independent outlets in order to afford opportunity for a more rapid mixing with the water. The question of interception to one or a few outlets, as at Buffalo or Louisville, or a direct discharge at numerous points, as at New York and Philadelphia, is a local one, and must depend on the circumstances; but it may be said that the tendency is toward the former plan for large cities, so that it has generally come to be looked on as the proper procedure.

The question as to the value of float experiments in locating sewer outfalls is sometimes raised. The speaker believes them to be of great value as indicating the direction in which sewage may be expected to flow under similar conditions. It is true that a surface float furnishes no measure of bottom velocities, which may be quite contrary in

direction; that the course taken by one float may be quite different from that of one set adrift a few minutes later, although under conditions which appear to be identical; and that it is influenced more or less by the wind; but in answer it may be said: Mr. Allen.

First.—A surface float represents the motion of that portion of the current which comprises the greater part of the sewage, through the tendency of the latter to rise and spread out over the surface, diffusion in a vertical direction being comparatively slow; and if bottom or mean velocities are desired these may be determined, approximately, by the use of double or rod floats.

Second.—Although the trace of a single float will not represent a true average result, yet it will show the path that sewage discharged at the starting point might take; and from a sufficient number of records the probability of its going in any given direction may be predicted with considerable certainty.

Third.—A float is influenced by the wind; but this, with properly designed floats, is mainly through surface currents induced by the wind, and it is the trend of the surface currents which determines the path that sewage would follow. The float, therefore, gives the desired information.

In questions involving dilution, depending on the net discharge of tidal and upland waters, the speaker does not consider results from floats as reliable as those obtained by current meters, as it is difficult, by means of the former, to integrate the motion of the filaments properly in a vertical section. Even with the current meter it is a long and expensive operation, if reliable results are to be obtained on tidal streams. An ingenious apparatus for facilitating such work was devised some years ago by Mr. Luigi d'Auria, and used by him on some public work undertaken under the direction of the late William Ludlow, M. Am. Soc. C. E. This device consisted of a pole which could be lowered vertically to the bottom of the stream from the side of a pontoon. An endless cord which passed through pulleys near the top and bottom of the pole was used to carry a ball of half the specific gravity of water to the bottom. At a given signal the ball was released, and, rising to the surface at a speed which a computation showed would be uniform, and then being caught by a floating grillage, the mean velocity was obtained by dividing the measured horizontal distance traversed by the time taken to rise to the surface.

Continuous records of single floats covering several tides are valuable in determining the range of tidal oscillation and the resultant movement seaward, but, to secure information regarding the probable dispersion of sewage, the plan used by the speaker, on Chesapeake Bay, of setting a large number of floats adrift at short intervals and locating them as frequently as practicable—each being identified by a number—is an excellent one to adopt, being rapid and economical.

Mr. Allen.

The importance of a careful consideration of local rainfall records is well illustrated in the case of San Francisco, where, if Eastern practice had been blindly followed, the size and cost of the sewers would have been greatly increased. In this respect it would be interesting to learn why the particular form of rainfall formula was adopted, and whether the more usual and simpler form, $I = \frac{a}{t+b}$, would not have answered equally well.

In forecasting the flow of sewage from a given district, the infiltration of ground-water is often an important but uncertain factor. The daily volume has been stated variously in gallons per mile of sewer, per acre or square mile drained, per capita of population, per linear foot of sewer joint, and per square foot of surface. The rational way is to consider infiltration as a function of the sewer surface, if of continuous concrete or brick, and of the linear foot of joint in the case of pipe or other sectional forms of construction.

The perviousness of the soil affects the rate of leakage to sewers. In New Orleans, George G. Earl, M. Am. Soc. C. E., Superintendent of the Sewerage and Water Board, found that:

"In open and unpaved areas we are probably getting double the amount estimated at times of very heavy rains, particularly when they follow a long dry spell, which has cracked open our clay soil, with deep and frequent fissures.

"Whether this flow is all seepage or partly due to flooded surfaces where manholes exist and where, even, some one may open a manhole cover occasionally, we cannot tell as yet * * *."

The allowance for infiltration was 0.003 sec-ft. per acre or 31 800 gal. per mile per day, and an estimate made after construction showed this figure—which is the maximum used for San Francisco—to be substantially correct.

In studies made for the sewerage of Baltimore, in 1897, Messrs. Hering and Gray, the Consulting Engineers for that work, recommended an allowance ranging from 0.00024 to 0.0018 sec-ft. per acre.

J. N. Hazlehurst, M. Am. Soc. C. E., secured a somewhat exceptional result* with an 8-in. vitrified pipe laid in 2½-ft. lengths, with a mortar joint of 1 part Louisville cement and 2 parts sand. This was kept covered for 6 hours with a damp cloth, and then immersed for 24 hours. The infiltration was then found to be only 1 329 gal. per mile per day.

X. H. Goodnough, M. Am. Soc. C. E., reported† that on 137 miles of pipe sewers in Massachusetts, varying from 8 to 30 in. in diameter, the infiltration was about 40 000 gal. per mile per day before connections were made, and that it amounted to 70 000 gal. per mile per

* *Engineering News*, August 27th, 1903.

† *Engineering News*, August 27th, 1903.

day on 700 miles of sewer, but that 80 000 gal. per day is estimated in the extensions of the sewers of the Metropolitan District of Boston. Mr. Allen.

In Malden, Mass., the infiltration was 50 000 gal. per mile per day, 70 gal. per capita per day, or 0.015 sec-ft. per acre.

D. H. Maury, Jr., M. Am. Soc. C. E., mentions experiments made in Peoria, Ill., on a 24-in. vitrified conduit laid in 2-ft. lengths, with 1:1 Portland cement mortar joints, in which the infiltration found by numerous measurements was generally from 8 000 to 35 000 gal. per mile per day, although, in some instances, the figure exceeded 60 000 gal.

The amount of infiltration, of course, will depend largely on the ground-water level, 30 000 to 50 000 gal. per day for each mile of well-laid pipe sewer being probably a fair allowance under ordinary circumstances, but where the exclusion of ground-water is important, this may be reduced by special precautions, and where the sewer lies above the ground-water level there will be a leakage from instead of into the sewer.

Alexander Potter, Assoc. M. Am. Soc. C. E., has secured good results by making joints with sulphur, sand, and pitch, and the speaker has done this by the use of tar pitch and cement, kneaded by hand to the consistency of dough, as has been the practice for a number of years, on the construction of sewers in Atlantic City. Some 20 years ago the speaker attempted to secure a water-tight joint with natural (Black Diamond) cement under furnished specifications, but he was unsuccessful. More recently, however, he has laid about ½ mile of 6-in. and 8-in. pipe, from 0 to 9 ft. below ground-water level, under the usual specifications of Portland cement mortar with a yarn gasket, which, after repairing about a dozen small leaks, the contractor made practically water-tight. To secure impervious work in this way, however, is more difficult and costly than it should be, and the speaker believes that some bituminous joint, poured like a lead joint, may eventually supersede cement—as it already has done to a certain extent abroad—and will furnish a more elastic joint, and, at the same time, a more economical result.

In the case of brick sewers, infiltration varies greatly with the quality of the work and the depth below ground-water level, but there are few definite data to serve as a guide.

Owing to the general substitution of concrete for brick in larger sewers, there is less infiltration than formerly, and this may be done away with practically by careful grading of the aggregate, wet mixtures, and continuous work. In the speaker's experience the greatest difficulty is at the springing line, where the joint between arch and invert, in spite of the bonding key, offers the least resistance to the entrance of external water under pressure. Infiltration in concrete sewers has been prevented entirely by water-proofing laid between the

Mr. Allen. concrete and a brick lining, but the extra cost of the water-proofing and the lining is hardly warranted in order to prevent the comparatively small amount of leakage which would otherwise occur. In a reinforced concrete sewer built at Waterbury, Conn., the infiltration under a head varying from 4 to 12 ft. was found to be 2.08 gal. per day per square foot of surface.

The poor alignment, size, and shape mentioned by the author, as existing in the old San Francisco sewers, have been similarly noted in Boston, New York, and Baltimore, and may be found in most cities large enough to have had sewers more than 50 years old. As a rule, smaller municipalities are freer from such absurdities; but, as an adopted standard design, the old San Francisco 3- by 5-ft. intersection, constructed uniformly 10 ft. below the surface, irrespective of topography or connecting grades, is unique.

The author alludes to changes in the elevation of bench-marks. This, to the extent of 0.10 ft. or more, may be due to upheaval by frost, or by a lowering of the ground-water level on account of the construction of drains, the sinking of wells, etc., depending on the character of the soil, unless the foundation extends to bed-rock. Aside from this, variations have been known to occur, due to more general movement of the earth's crust. Large areas are known to be involved in these movements, such as the gradual sinking of the New England Coast, which, as John R. Freeman, M. Am. Soc. C. E., has pointed out,* is taking place at the rate of about a foot a century.

But there is reason to believe that there may be other alterations in the relative elevations of points within such short distances of each other as to affect the bench-marks of a municipality.

Some time ago, Horace Andrews, M. Am. Soc. C. E., brought a case in point to the attention of the speaker in connection with the "re-levelling of a number of points of the Bavarian precise levels which were originally determined in the interval between 1869 and 1873 and the mean levels taken in 1897."† These were both levels of the highest order, with refinements as to accuracy of standard and permanent marking of bench-marks, which, unfortunately, are often disregarded.

"The discrepancies revealed were about equally plus and minus and in only one or two cases did there seem to have been some gross change in the foundation of the building on which the mark was placed.

"The differences in this Bavarian work were found by touching on nearly fifty points scattered over a large region. * * * Discrepancies of from 13 to 14 mm. seemed to be common, while several were as large as 15 to 17 mm. and two were of 41.5 mm. and one was 86.7 mm. In addition to these was the gross change alluded to of 233.5 mm. in one point."

* Report on the Charles River Dam, 1905.

† *Zeitschrift für Vermessungswesen*, August 1st, 1900.

Mr. Allen.

About 1894, precise levels were run in connection with a general topographical survey then being made of Baltimore by Colonel H. T. Douglas, and numerous bench-marks were established. Twelve years later the speaker had occasion to test a good many of these, and, although precise levels were not run, the discrepancies noted were checked so carefully that he was convinced that numerous changes had occurred during the intervening years. This opinion was confirmed by results reached on another division of the same work, where it was observed that the greatest discrepancies occurred where the slopes were most abrupt. Discrepancies of this kind might appear to be due to a difference in the graduation of the rods, but as these had been compared carefully with a standard, the conclusion reached was that this was not the cause. The apparent errors of elevation noted varied up to several hundredths of a foot.

It would be interesting to learn the general experience of municipal engineers who have had the opportunity of checking benches established by precise methods after the lapse of several years.

MEMOIRS OF DECEASED MEMBERS

NOTE.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

GEORGE ADGATE, M. Am. Soc. C. E.*

DIED JANUARY 23D, 1909.

George Adgate was born on September 28th, 1845, at Keeseville, New York, and was the eldest of a large family of children. His ancestors emigrated from England to Massachusetts and Connecticut during the earliest Colonial times. Through a marriage between the son of the first Adgate who came to this country, and a granddaughter of Elder William Brewster, the Adgates were placed in the line of descent from the Pilgrims.

The family has been useful and influential in public affairs throughout its history. Mr. Adgate's great-grandfather, Judge Matthew Adgate, was a member of the New York Legislature and of the convention which formulated the first constitution of the State of New York. His grandfather and father were also members of the New York Legislature.

After his early education at Keeseville Academy, one of the best of the old-time schools, Mr. Adgate spent two years keeping a store in the oil regions of Pennsylvania. Later he became connected in business with his father-in-law, Mr. Whitney, a contractor on Lake Champlain. The work consisted principally of dock-building, much heavy construction of that kind being necessary in those days, before any railroads had been built on the shores of the lake. The fifteen years he spent in struggling with the difficulties of subaqueous work, often under severe climatic conditions, were instrumental in developing his judgment and resourcefulness.

In 1882 Mr. Adgate was engaged by William Sooysmith and Son as Manager of bridge foundation construction, most of which involved the use of pneumatic caissons. He was engaged with this firm and its successors throughout the remainder of his busy life. Here his experience and sound judgment were often tested while building foundations in large rivers where varying conditions as of floods and ice gorges had to be anticipated and provided for in advance. In this warfare with the elements it can be truthfully said that he never lost a battle.

He was in active and responsible charge of the building of the foundations and piers of the following bridges:

*Memoir prepared by E. L. Abbott and Daniel E. Moran, Members, Am. Soc. C. E., and Edwin S. Jarrett, Assoc. M. Am. Soc. C. E.

Terre Haute, Ind. "Big Four" Bridge over the Wabash River.
Van Buren, Ark. Bridge over the Arkansas River.
Needles, Cal. Bridge over the Colorado River.
Cuivre Junction, Mo. Bridge over the Cuivre River.
Boone, Iowa. Chicago and North-Western Bridge over the Des Moines River.
Harvey, Iowa. Chicago, Rock Island and Pacific Bridge over the Des Moines River.
Kansas City. Chicago, Rock Island and Pacific Bridge over the Kaw River.
Clinton, Iowa. Chicago and North-Western Bridge over the Mississippi River.
Ottawa, Can. Interprovincial Bridge over the Ottawa River.
Glasgow, Mo. Bridge over the Missouri River.
East Omaha, Nebr. Bridge over the Missouri River.
Plattsmouth, Nebr. Chicago, Burlington and Quincy Bridge over the Missouri River.
Sibley, Mo. Bridge over the Missouri River.
Sioux City, Iowa. (Pacific Short Line) Bridge over the Missouri River.
Arthur City, Tex. Bridge over the Red River.
Gainesville, Tex. Bridge over the Red River.
Fort Worth, Tex. Gulf, Colorado and Santa Fé Bridge over the Trinity River.
Marietta, Ohio. Bridge over the Ohio River.

He was also in charge of many smaller bridges.

Mr. Adgate was associated in the construction of the foundations of the Baltimore and Ohio Bridge across the Susquehanna River, at Havre de Grace; he also supervised the sinking of several deep shafts and the building of numerous docks, dams, intake piers, lighthouses, and other foundation work. At the time of his death, January 23d, 1909, he was Western Manager for The Foundation Company of New York City.

Mr. Adgate inherited a wholesome and rugged constitution, which enabled him to contend successfully with the many adverse conditions and surroundings incident to his work in the difficult kinds of construction on which he was continuously engaged.

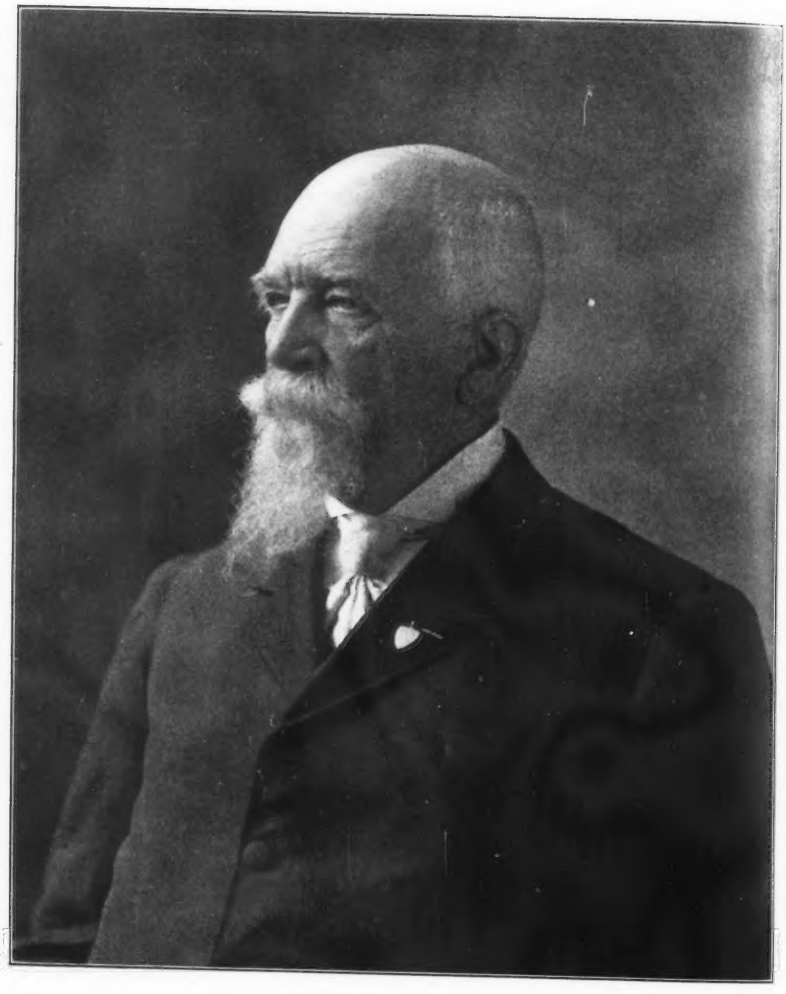
His organizing ability was wonderful, and his keen foresight, calm judgment, and iron determination were chiefly instrumental in the success of the many works for which he was responsible.

Personally, Mr. Adgate was kind, just, and lovable. He could direct the operations of others so as to obtain their best efforts and loyal support, and the engineers and associates with whom he worked, had, to the last, the highest regard for his ability and integrity. He

had a fund of quiet humor, which came to the aid of his associates and helpers in the most trying situations and lightened their load. He was always ready to help young men after they had shown their worth, but believed in trying them out in a rough but kindly way. He was genial and bluff, but on questions of principle stood like a rock against persuasion or threat.

He was married on September 19th, 1866, to Miss Martha Whitney, who survives him, together with a daughter, Mrs. J. R. Dean, of New Britain, Connecticut, and a son, Frederick W. Adgate, Assoc. M. Am. Soc. C. E., of Chicago.

Mr. Adgate was elected a Member of the American Society of Civil Engineers on April 1st, 1896.



WILLIAM PRICE CRAIGHILL

WILLIAM PRICE CRAIGHILL, Past-President, Am. Soc. C. E.*

DIED JANUARY 18TH, 1909.

A long and active life spent in the service of the country builds its own enduring monuments. Little remains to his friends other than to pen a few inscriptions, which, though inadequate, may serve to remind the passer-by more clearly of the man whose work is there in testimony of the duty done and the good accomplished. An engineer, eminent in his profession, a Past-President of the American Society of Civil Engineers, and for a time the head of his Corps in the Army, must necessarily have such monuments widely scattered over our land. It is so with General Craighill. The records of his work are found in our improved rivers and harbors, in our fortifications, and also in the lives of the younger men of the profession who were associated with him and whom it was his delight to help.

William Price Craighill was born in Charlestown, West Virginia (then Virginia), on July 1st, 1833, the eldest son of William Nathaniel and Mary Elizabeth (Brown) Craighill. He was of mixed English and Scotch blood, and his ancestors bore a prominent part in the early history of Virginia. The earliest of the American Craighills, William, is believed to have been born near Carlisle, England. His son, Nathaniel, served in Captain William Washington's company of Minute Men, in 1774-75, and later was in the Continental service. The first William Price Craighill, son of Nathaniel, was born at Craighill Point, Westmoreland County, Virginia, served as a volunteer in the War of 1812, and was a member of the Legislature of Virginia about 1817. General Craighill, his grandson, was named after him. Among other ancestors can be named the Littles, one of whom was a friend and neighbor of General Washington at Mount Vernon, and one of his pall-bearers; the Browns, who claim descent from one of the founders of Jamestown; and the Rutherfords, of an ancient Scottish family, one of whom, Robert, the maternal great-grandfather of General Craighill, was also an associate and friend of Washington, as a surveyor and fellow-officer of the early Indian wars, and a participator in the political agitations in Virginia immediately preceding the Revolution. He held many places of trust in Virginia, and was the first Member of Congress elected from the west of the Blue Ridge, having represented Berkeley County from 1793 to 1797.

Among the founders of the Charlestown Academy, in 1795, were three of the grandfathers of General Craighill, Little, Craighill, and Rutherford. Here the General received his early schooling, following his grandfather and father, and here his sons succeeded him. General

* Memoir prepared by the following Committee: General Charles W. Raymond, Colonel William M. Black, Lieutenant-Colonel Gustave J. Fiebeger, Bernard R. Green and John Thomson, Members, Am. Soc. C. E.

Craighill was for years a Trustee of the school, and finally President of the Board. On July 1st, 1849, at the age of sixteen, he became a Cadet at the United States Military Academy, the youngest of his class, the roll of which contained many names destined for fame, and among these may be mentioned McPherson, Schofield, Sheridan, Sill, Sooy-Smith, Terrill, Vincent, Tyler, Sweitzer, and Hood. He was graduated in 1853, second in the class, which McPherson led, and was commissioned Brevet Second Lieutenant, Corps of Engineers. His subsequent advances in rank in the Corps of Engineers of the Army were as follows: March 3d, 1855, Second Lieutenant; July 1st, 1859, First Lieutenant; March 3d, 1863, Captain; November 23d, 1865, Major; January 2d, 1881, Lieutenant-Colonel; January 10th, 1887, Colonel; May 10th, 1895, Brigadier-General and Chief of Engineers. On March 3d, 1865, he was made Brevet Lieutenant-Colonel "for faithful and meritorious services during the war, and particularly for services in the defence of Cumberland Gap, and the ulterior operations of General Morgan's forces." He also received the brevet of Colonel "for gallant and meritorious services during the rebellion," but this he declined.

When the Legislature of Virginia adopted the ordinance of secession, in 1861, Lieutenant Craighill was confronted by the grave question whether he should follow the fortunes of his native State or remain in the service of the Union. On the one hand were all the associations of his youth, the pride of his ancestry, and the certainty of immediate professional advancement; on the other hand, a solemn sense of duty and the sincere belief that the ultimate welfare of the whole country depended on the success of the cause of the Union. After the most anxious and thoughtful consideration, he decided that it was his duty to remain in the service of the Union; but he refused to seek advancement in a war in which he found himself opposed to his own people, and thus voluntarily surrendered the opportunity of obtaining the high honors and personal distinctions which were attained by so many of his associates of similar age, training, and experience in both the Confederate and the Union armies.

From 1853 to 1856, Lieut. Craighill was on duty on the improvement of the Savannah River and in the construction of fortifications at its mouth, on fortification work at Dry Tortugas, and in fortification construction and harbor improvements in Charleston Harbor. He was stationed at Fort Sumter during the yellow fever epidemic which ravaged Charleston in 1854-55, and while there became cognizant of one of the terrible abuses which slavery made possible, which made on his mind an indelible impression of the evils of the system. In 1856, he went to Washington as Assistant to the Chief of Engineers, General Totten, Hon. M. Am. Soc. C. E. In later life, he was fond of recalling the methods of that talented and painstaking official, who, with his own

hands, frequently made working drawings of the details of the beautiful stonework to be found in the fortifications of that date, showing a possibility for the careful individual study of the problems of the office, which must be a cause of envy for General Totten's overworked successors of later days. In 1859, Lieut. Craighill was ordered to the Military Academy as Principal Assistant Professor of Engineering, and remained on that duty until June, 1863. While at the Academy, he served for two years as Treasurer, and for one year was in command of the detachment of Engineer soldiers at West Point. He also compiled "The Army Officer's Pocket Companion," a treatise which proved very useful to the Volunteer Army, and translated Dufour's "Cours de Tactique" and, jointly with Captain Mendell, Jomini's "Précis de l'Art de la Guerre."

In the summer of 1862 he was Chief Engineer of the Division of General George W. Morgan, Army of the Ohio, and was engaged in the defense of Cumberland Gap. In his report on the campaign, dated October 12th, 1862, General Morgan writes:

"Nor can I close this report without calling the attention of the Commanding General to the important services rendered me by Lieut. W. P. Craighill of the Engineer Corps. He is an officer of distinguished merit, and is thoroughly informed on all subjects connected with the art of war. He would make an able Chief of Staff, or fill with high credit any other position to which he may be assigned, and deserves a much higher grade than he now holds."

In the summer of 1863, Captain Craighill was Chief Engineer of the Department of the Monongahela, and was engaged in the construction of the defences of Pittsburg, then threatened by the raid of General John Morgan. Later, he was Assistant Engineer in the construction of the defences of Baltimore, and in 1864, was Chief Engineer of the Middle Department and Eighth Army Corps. In 1864, he again went to the Military Academy, where he served as Assistant Professor of Engineering, Instructor of Practical Military Engineering, in command of the Engineer Detachment, and as Treasurer, serving also on Boards for the defences of San Francisco, for fortifications at Willets Point, and for New York.

In November, 1865, he was sent to the Baltimore District, in charge of the construction of the fortifications and of river and harbor improvements, and, with the exception of the tour of duty from 1866 to 1870 as Assistant to the Chief of Engineers, General A. A. Humphreys, Hon. M. Am. Soc. C. E., he remained on that duty until promoted to be Chief of Engineers in 1895. His fortification work included the construction of works at Baltimore, Washington, and Hampton Roads. The river and harbor works extended from the Susquehanna to the Cape Fear, and as far west as the Great Kanawha, in West Virginia. In Baltimore Harbor, the dredged channel constructed by him bears his name.

The improvement of the Cape Fear River presented a notable engineering problem, and the successful closing of the false mouth across the shifting sands of the coast was a work of great importance. In this work was evolved the log and brush mattress, used extensively later on the South Atlantic Coast. Perhaps the most notable and successful of the works was the canalization of the Great Kanawha River, in which the Chanoine system of movable dams is used. It is believed that the introduction of this system in the United States is mainly due to the late Colonel Merrill, with whom Colonel Craighill visited Europe in 1878, for an inspection of works of that character; but the first Chanoine dams actually constructed and placed in service in the United States were on the Great Kanawha River, under Colonel Craighill.

In addition to these regular duties, Colonel Craighill served on many boards formed for the consideration of projected improvements, among which may be named those for the Columbia, Mississippi, Hudson, Ohio, Delaware, Potomac, Cape Fear, Great Kanawha, James, and other rivers, and the breakwaters for Delaware Bay, Sandy Bay, Massachusetts, and San Pedro, California. For several years he was a member of the Light-House Board and of the Board of Engineers in New York, to which were referred questions relating to fortifications and river and harbor improvements.

In 1884 the system of grouping a number of river and harbor districts into a division was inaugurated. From that date until 1895 Colonel Craighill was the Division Engineer of the Southeast Division. When this division was first formed, it had several of the younger officers of the Corps of Engineers as its district engineers, and was known as "the kindergarten." The Division Engineer's visits were always welcome, and he took a keen interest in the work of the younger men, approving their methods whenever practicable. "For," he said, "I have found that generally there are several ways of accomplishing a given result, and that it is best to follow the plan of the man who is to do the work providing that plan be sensible"—a truth not always appreciated by superiors. At another time, he said to a young officer who was reporting to him for duty, "Mr. ———, I propose to be the laziest man in this district, and do not propose to do anything that I can have done by one of my assistants." It is needless to say that, under such a man, the assistants always did their best.

As Chief of Engineers, it fell to General Craighill to arrange for the prompt construction of the many new works of defence for which Congress had for the first time for years made large appropriations. Under him, a number of changes were made in the methods and policy of the Engineer Department, the most important of which was the inauguration of the practice of having the type plans of the batteries, with the general project for the defense of a harbor, referred to the

local engineer, prior to final adoption, in order to obtain the benefit of his local knowledge and experience. Criticisms of the plans were requested from the various branches of the service interested. The results were immediate and beneficial. Due to his insistence, the development of the disappearing carriage for 12-in. guns was hastened by the Ordnance Department, thus permitting the abolition of the very expensive lift type of battery, which, before that, was the only type for 12-in. guns affording protection while they were being loaded. He also changed the type of the mortar batteries, and had important improvements made in the submarine mine system. Under him, was begun the practice of disseminating important technical information among the officers of the Corps by means of mimeographs.

Incident to his service, General Craighill had to appear frequently before the various committees of Congress, where he made many friends. His succinct and straightforward answers to questions were always received with marked attention. His sound judgment and evident thorough knowledge of the subject at issue caused his opinions to have great weight. An evidence of this is found in the history of the Washington Aqueduct tunnel. This had been abandoned after the discovery of fraudulent work on the part of the contractors, and the tunnel project had become thoroughly discredited in Congress. After an expert examination into its merits, General Craighill presented to Congress a full and convincing report, and, as a result, the necessary appropriations were made and the work was successfully carried to completion. The tunnel to-day forms an important link in the Washington water-supply system.

On February 1st, 1897, General Craighill was retired from active service at his own request. Excepting a limited service as a member of the Board of Consulting Engineers of the Dock Department of the City of New York, his retirement ended his active career as an engineer. He died at Charlestown, West Virginia, on January 18th, 1909. The immediate cause of his death was stated by his physicians to have been "disability following a stroke of apoplexy causing paralysis of the right side."

General Craighill was married on October 14th, 1856, to Mary A. Morsell, daughter of Judge James S. Morsell, of Washington, D. C., and, after her death, was married, on September 22d, 1874, to Rebecca Churchill, daughter of the Reverend Alexander Jones, of Virginia. He had three sons and three daughters. One of the sons, William E. Craighill, was graduated second in the class of 1885, United States Military Academy, and is now a Major of the Corps of Engineers, U. S. Army.

Of civic duties General Craighill did his rightful share. The Episcopal Church, of which he was a life-long member, showed its appreciation by making him its deputy from the Diocese of West

Virginia to ten successive Triennial General Conventions, beginning with that of 1880, and also a delegate to the Pan-Anglican Conference in London in 1908. He was a member of the Malta Lodge of Masons, Charlestown, West Virginia, from July, 1855, to his death. In 1888 he became a member of the American Association for the Advancement of Science. He was, in 1895, the sole honorary member of the Historical Society of Maryland. In 1897 the Washington and Lee University conferred on him the degree of LL.D. The following offices were offered to him at various times and declined, viz.: Commandant of Cadets at the Virginia Military Institute in 1860; charge of the Water Department of the City of Baltimore under Mayor Malsted; Superintendent of the United States Military Academy, once by General Sheridan and once by General Schofield; President of the University of West Virginia, in 1895; Superintendent of the Coast and Geodetic Survey, by President Cleveland, and membership in the Isthmian Canal Commission, by President McKinley.

William Price Craighill was elected a Member of the American Society of Civil Engineers on October 7th, 1885; he served as a member of the Board of Direction in 1892 and 1893; and was elected President in 1894, during his term of office as a Director; he was elected an Honorary Member on March 23d, 1896.

It is probably within the truth to say that no epoch in General Craighill's career gave him more pleasure and satisfaction than his official connection with the American Society of Civil Engineers, first as a Director and finally as its President; and it certainly is a fact that this connection with the Society resulted largely to its good, and brought about many permanent and delightful friendships.

In connection with the work of the Society, he was particularly attentive to apparently trivial details, that is, when such details appeared to have an ultimate bearing of importance; on the other hand, when no subsequent advantage was apparent, he was perhaps unequaled in his ability to "side-track" such matters, without, however, appearing brusque or offensive in manner. As is frequently the case where routine matters must be passed upon by a board composed of many men, the tendency is to fritter away time to no useful purpose. Early in the General's service as a Director it was made evident that he had little in common with such discussions, and could confidently be counted on to "vote first and talk afterward." Therefore, as might be expected, he was a business executive of the first order, and it was a delight to sit with him in the Board meetings of the Society.

As a presiding officer, he was exceedingly efficient, combining in a rare degree, seldom equaled, affability, tolerance, tact, dignity, retentiveness of memory, and a swift comprehension of a controverted point. A decision never appeared to be made too quickly; nor was there any uncertainty when rendered.

It is believed that no officer of the Society made, in so few years, so many friends among the younger members of the Society; for, to the young engineer, his courteous and kindly attention was equally a delight to the recipient and to the observer.

He was in all respects well-rounded—a man of many parts. Although bound by the traditions of many years of military service, he was yet peculiarly open-minded in recognizing the necessary differences which have existed, and must exist, in the performance of engineering work under private or individual control and Governmental control. He undoubtedly possessed the attributes essential to success in any walk of life, wherein he would as surely have made his mark as he did in that branch of the Engineering Profession which he adorned so conspicuously.

General Craighill was a member of various committees of the Society, and it may serve to illustrate both the simplicity of his character and his keen discernment of a useful purpose to mention his connection with the Committee which designed the present Society badge. Here, on the face of the matter, the duty was of merely nominal importance, yet he devoted much thought to the subject, and made many suggestions. It is well remembered how, at one of these meetings, he referred with evident gratification to his West Point class pin, the class of 1853, which bore the device of a mortar, with a triangular pile of balls stacked in front of it, and over all the words, "We separate for service." The point he was making was that he believed this sentiment, in his own case at least, had proved to be a potentiality for good, and consequently the same might result from the work in hand. After the present plain lettered badge had been adopted by the Society, he took the keenest interest in having the decision of the Committee carried out, that it should be produced as nearly mechanically perfect as would be possible by available processes and manual skill.

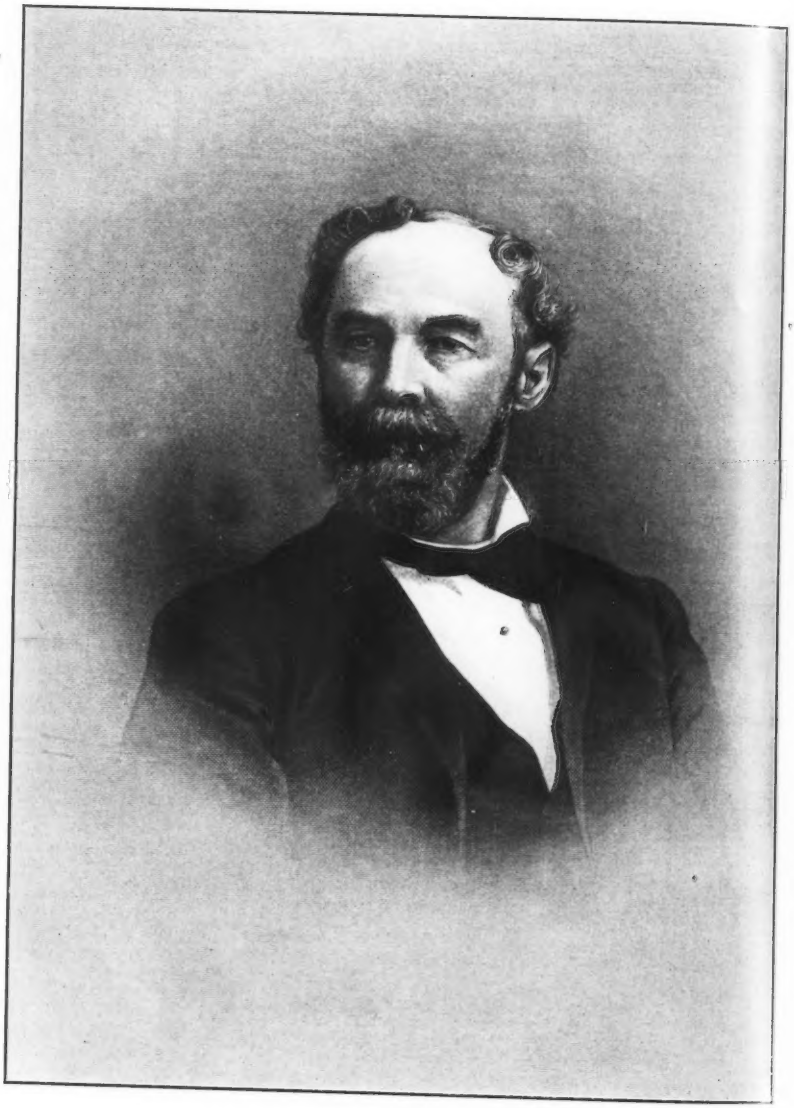
Thus it can verily be believed that in all matters, great or small, his heart was bound in the Society to the end that its influence and prestige might be broadened, the position of the engineering profession be enhanced, and that we, who "separate for service," might with pride and honor carry its insignia of membership.

This brief and all too inadequate tribute to the General's association with the Society may be best closed by his own estimate of the honors it had conferred upon him, for, by his expressed instructions, the following inscription is to be cut upon his tombstone:

WILLIAM PRICE CRAIGHILL,
Brigadier-General and Chief of Engineers,
United States Army;
Honorary Member and Past-President,
American Society of Civil Engineers.

His was a true and loving heart, strong with human feeling. A fitting crown to such a life and character is enduring memory among one's friends and associates. It is certain that he himself would have preferred no honor to his memory more acceptable than this, which, and especially from the members of the Society who had the pleasure and honor of his acquaintance, is abundantly assured.

"Such graves as his are pilgrim's shrines,
Shrines to no code or creed confined."



Chas Downey

CHARLES HERMANY, Past-President, Am. Soc. C. E.*

DIED JANUARY 18TH, 1908.

Charles Hermany was born in Lynn Township, Lehigh County, Pennsylvania, on October 9th, 1830. He was the son of Samuel and Salome Wannemacker Hermany. His ancestors were of Swiss, French, and English stock, who had emigrated to the United States as early as 1700, and settled in the southeastern part of Pennsylvania. They were agriculturists, and, with their descendants and contemporaries, developed the district from a wilderness to ideal farming land. The farm on which Mr. Hermany was born still belongs to the family, having been deeded to his ancestor, George Philip Hermany, by William Penn.

Mr. Hermany received his early education in the public schools of his home town, where he developed a decided taste for mathematics. Having determined to adopt the Profession of Civil Engineering as his life work, he entered Minerva Seminary, at Easton, Pennsylvania, and was graduated therefrom in two years. Financial conditions prevented him from continuing his collegiate education, and for three years he taught school, at the same time continuing his studies in mathematics and in his chosen profession with a determination which characterized all his work in later life.

In May, 1853, he went West and found employment in the City Engineer's Department of Cleveland, Ohio. Here he was engaged for four years, gradually advancing to the position of First Assistant to the City Engineer, the late T. R. Scowden, M. Am. Soc. C. E. When, in 1857, Mr. Scowden, as Chief Engineer of the Louisville Water Company, took charge of the construction of the water-works system in Louisville, Kentucky, Mr. Hermany went with him as his Principal Assistant. In this capacity he served during the entire construction period and until the works were completed and put in operation in October, 1860. On January 1st, 1861, Mr. Hermany was appointed Chief Engineer and Superintendent of the water-works, which position he held continuously up to the time of his death.

The growth of the city soon proved that the works, as first constructed, were entirely inadequate, and this necessitated their general enlargement. To Mr. Hermany was assigned the duty of designing and constructing a new pumping station on the river bank, which would secure an abundant supply from the river, the stages of which varied more than 40 ft. between high and low water. The treacherous condition of the ground necessitated the sinking of a pneumatic caisson, in

* Memoir prepared by the following Committee: G. H. Benzenberg, Past-President, Am. Soc. C. E., R. Montfort, and Webster Gaziay, Members, Am. Soc. C. E.

order to secure a firm foundation for the pump-house and the pumping engine, as well as to provide for an inlet chamber. The contract was let to an experienced and responsible firm, which, however, finding the work of a hazardous nature, refused to proceed with it. This condition obliged Mr. Hermany to prosecute the work by day labor, requiring his personal supervision and utmost skill for a number of years. In this work he was successful, as he was also in the installation of pumping machinery designed principally by himself, which, at the time of its test run, attracted general attention because its high economic efficiency exceeded all known records. In addition to a betterment of the distribution system, he designed and constructed the Crescent Hill Reservoirs. His last great work was to design and construct the filter plant, including the covered clear-water basin and the high-service pumping station, with its equipment of pumping machinery.

Being extremely careful and prudent, Mr. Hermany conducted a series of exhaustive experiments, lasting more than a year, in order to determine the best method of treating the river water in the process of proposed purification. Very particular in all matters, he practically worked out every detail of all the work which he designed and constructed, the specifications which he prepared for his large pumping engine being an evidence of the minutiæ to which he carried his work. As exacting as he was in these matters, so was he in his requirement of the fulfilment of every obligation on the part of contractors. Faithful to every duty, he insisted upon the same compliance from everyone else.

As an engineer Mr. Hermany was resourceful in every emergency, which enabled him to meet successfully any contingency, as he often had occasion to demonstrate, perhaps at no time so signally as when he promptly restored the works to service by converting the remnants of a tall stand-pipe into a great air chamber, after it had been demolished by a disastrous tornado which swept over Louisville in 1890.

These qualities, as well as his skill, attained for him an eminence in his profession which resulted in his counsel and advice being sought by the water departments of many cities throughout the country. He designed the water-works at Frankfort, and at Bowling Green, Kentucky, and the pumping station at Evansville, Indiana. He also served, in different capacities, the water-works of Nashville, Memphis, St. Louis, Kansas City, Milwaukee, Providence, Chicago, and, finally, as Consulting Engineer during the construction of the new water-works of Cincinnati, Ohio.

Mr. Hermany possessed administrative abilities to a very high degree, which he manifested in conducting the office of Superintendent of Water-Works for more than forty-seven years to the perfect satisfaction of the Water Board and of every water taker, the confidence and regard of all of whom he always possessed.

His reputation for honesty and integrity was ever proverbial, and though, at times, he was impatient and blunt when he suspected unfairness or deceit—for he hated all sham and sordidness—he was at heart scrupulously just, as well as kind, generous, and thoughtful. Courteous and affable to everyone, he was deeply true and loyal to those who were privileged to be enumerated among his friends. The great affection and high esteem in which he was held by the members of his profession, his fellow-citizens, and his neighbors was shown by the great number who were present at the last sad rites, by the immense number of floral tributes under which the casket was buried, and in the last words which were spoken over his remains:

"To discover one's talents, to find one's place in the economy of the world, to put one's powers to the highest use in the service of one's fellow-men, is to merit the approbation of the Judge of all. This is what Mr. Charles Hermany did. He served this community with a faithfulness and devotion to duty that it would be difficult to equal and impossible to excel."

On December 19th, 1864, Mr. Hermany married Miss Sallie Adams, only daughter of Daniel and Sophia (Perdue) Adams. Forty-three years of happy life followed this union, during which seven children blessed their home, three of whom, Irene H., the wife of Mr. Owen Gathright, of Louisville, Emily H., the wife of Mr. James P. Donahue, of Davenport, Iowa, and Madeline, an unmarried daughter, together with his wife, survive him. During the last nine years Mr. Hermany lost his three sons and his youngest daughter, Hetty May, wife of Mr. James L. Smyser, of Louisville. These losses were severe blows to him, from the effects of which he never fully recovered.

Mr. Hermany's health was generally good, and he was ill for only a few days before his death. It had been his intention to attend the Annual Meeting of the Society in January, 1908, but, a few days before, he contracted a bad cold which rapidly developed into pneumonia, involved both his lungs, and resulted in his death within a very few hours, he having been confined to his home less than three days and to his bed less than one day.

A sad feature in connection with his death was the fact that, after many years of earnest and perplexing labor, he was about to complete the filtration works, and this was to round out his professional labors after half a century of faithful, zealous, and devoted services to his home city.

Mr. Hermany was a remarkably self-reliant man, and always exhibited a consciousness of faith in his judgment when once he had arrived at a conclusion. He was a constant reader, and was endowed with splendid intellectual gifts, possessing a mind which was quick to discern and independent in its deductions.

Mr. Hermany never held any political office, though he was always

deeply interested in questions which affected the welfare of his country, state or city, and he did not hesitate to give expression to his views. He was a loyal member of the Unitarian Church of the Messiah, and, for thirty years, was a member of its Board of Trustees. He was the first President of the Engineers' and Architects' Clubs of Louisville, and was re-elected for five consecutive terms.

On January 6th, 1869, Mr. Hermany was elected a Member of the American Society of Civil Engineers. In November, 1879, he was elected and served as a member of the Board of Direction and, in 1891, as Vice-President. In 1904, he was elected President of the Society, and presided at the meetings of the International Engineering Congress, held in St. Louis during the Louisiana Purchase Exposition. During this Congress he was elected an Honorary Member of the Institution of Civil Engineers of Great Britain.

He was also a Member of the Franklin Institute, and of the American Water-Works Association, as well as of the Salmagundi and Filson Clubs, of Louisville, the leading literary organizations of that city.

WALTER LORTON DUSENBERRY, Jun. Am. Soc. C. E.*

DIED MAY 13TH, 1909.

Walter Lorton Dusenberry, the son of Edmund and Caroline Elizabeth (Bennett) Dusenberry, was born in New York City, on February 9th, 1862. On his father's side Mr. Dusenberry was descended from the Von Doesenbury family, who settled in New Amsterdam early in the seventeenth century. Members of the family served with distinction in the Revolutionary War. His mother's family was also of Dutch and Huguenot descent, having settled on Long Island two centuries ago.

Mr. Dusenberry received his early education at Anthon's Grammar School, in New York City. Afterward, he entered the School of Mines of Columbia University, from which he was graduated in 1884 with the degree of E. M.

For some time after his graduation, Mr. Dusenberry was connected with the United States Coast and Geodetic Survey, and was also engaged in mining engineering in the western part of the United States and in Mexico. In 1887 he received an appointment on the New Croton Aqueduct, in New York City, being promoted later for valuable services rendered. From 1889 to 1895 he served as Assistant Engineer in the Department of Public Parks. From 1895 to June, 1897, Mr. Dusenberry was employed, except for a brief interval, in the Department of Public Parks and the Department of Highways, of New York City, in charge of the opening and paving of streets. He was then appointed Engineer to the Commissioners of Accounts, and on leaving that office in 1903, he engaged in expert mining engineering work, and in the examination of properties in the Southwest and in Mexico.

Mr. Dusenberry was married on May 11th, 1899, to Nina Jay Wright, who survives him. He was a member of the Huguenot Society and the Sons of the Revolution, and was also a Mason. His personal character was of the highest, and he was devoted to his profession.

Mr. Dusenberry was elected a Junior of the American Society of Civil Engineers on November 5th, 1890.

* Memoir prepared by the Secretary from material furnished by Mr. Dusenberry's family.

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AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

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THE REDEMPTION OF THE GREAT VALLEY OF CALIFORNIA.*

By A. D. Foote, M. Am. Soc. C. E.

The Great Valley of California, as Nature built it, by filling an inland sea with débris from the hills, seemed capable of feeding the world with wheat fifty years ago, when men were reaping from its surface the cargoes for those "swift shuttles of an Empire's loom," the "Gypsies of the Horn."

Fifty years of mishandling natural riches and spurning natural laws have so far injured it that now it may be said, in an economical or engineering sense, the Great Valley is lost to the world. The historic wheat fields have deteriorated until they are no longer of appreciable value to the State. When Mr. Davis, after whom Davisville was named, sat on his housetop and, with a spy-glass, watched his plow teams crawl over his vast wheat fields, he expected a harvest of 40 bushels to the acre. A member of Balfour, Guthrie and Company, the great wheat buyers of the Pacific Coast, recently said that those same fields would now produce from 6 to 10 bushels only, and that of an inferior wheat. Even the mills are obliged to import wheat into California and mix it with the native crop to bring their flour up to grade. Fifty years of crops with no fertilization has made the Valley of almost no economical value.

The two great rivers, which should be 6 to 10-ft. channels, reaching from Red Bluff to Lake Tulare and from Marysville to the Bay, are

* This paper will not be presented at any meeting, but written communications on the subject are invited for publication with it in *Transactions*.

shallow sloughs through which a few flat steamboats and barges struggle from one sand-bar to another below Sacramento and Stockton. Fifty years ago they were of immense value to the community as commerce bearers, and, though not large enough to carry the highest floods, they were of vast importance in ameliorating them.

During the winter of 1908-09, it so happened that the first heavy rains fell on the water-sheds of the American and Yuba Rivers. The American rose until the levees on the east side of Sacramento were but 6 in. above the river surface, and the bridges of the Southern Pacific and Western Pacific Railroads went out, though the Sacramento River on the north side of the city was 2 ft. below the high-water mark. The heavy rainfall then shifted to the northern counties, and in a few days the Sacramento River was higher than ever before known, though the American River had gone down. Few realize what good luck it was for Sacramento City that the flood came down the American first. Had the heavy rains fallen in the northern counties first, the crest of the flood would have reached Sacramento about in time to meet the crest of that from the American River. The levees east of the city would have gone out, and the whole American River, swelled by the backwater from the Sacramento, would have swept the city, a mass of wreckage, drifting toward the Bay. A few of the stronger buildings might have stood, but probably the American River channel would now be in front of the Capitol.

The word "luck" is often used to cover ignorance. In this case it covers ignorance of the laws of rainfall. It is not known why the rains came to the American River first, but it is known that it is merely a question of time when the laws of rainfall will swing the rains to the northern counties first and the American afterward, and things will happen as above described.

There can be no reasonable doubt but that the floods in the Great Valley will continue at irregular intervals, and the time will come when the combination of the rain conditions will be such as to destroy, at different times, perhaps, every town and village on its lower levels. No levee system alone can be built that will withstand the great floods. As Mr. Ellery, State Engineer, in his last Report, so well shows, a channel large enough to carry the flood of the past winter between levees is an engineering absurdity. There must be some other method.

The Report of the State Engineer for 1907-08 discusses very ably the apparent antagonism of the different interests in the Great Valley —navigation, flood protection, drainage, irrigation, mining débris— and shows that work done for navigation alone is fatal to flood protection because it contracts the drainage channels in order to give depth at low water, and thus prevents the free passage of the floods. Works for irrigation alone take water needed for navigation. Mining is stopped because the débris fills the drainage channels and spreads over the farm lands. Drainage is blocked by the levee system built for flood protection; and to build levees for flood protection alone is hopeless. The State Engineer also very ably says, in seeeking a remedy: "The first requisite is a unification of purpose and harmony of effort among the varied interests involved."

"The solution of the problem for the tiller of the soil essentially solves the whole train of problems," were the words used by Dr. Chamberlain in his paper on "Soil Wastage," read at the second Conservation Conference of the Governors, at Washington.

It is believed that "the solution of the problem for the tiller of the soil" is in harmony with and solves all other problems in the Great Valley, and it is because this problem has been slighted or left out entirely that so little has been accomplished, and so many antagonisms aroused, in the attempts thus far made toward the conservation of its vast natural riches.

During the winter of 1908-09 a correspondent of one of the newspapers mentioned the flooding of 5 000 acres of wheat, but that the owner did not regret it much, as the next crop would be more than doubled by the fertilizer deposited by the water. In this simple statement lies the secret of the redemption of the Valley. It is no exaggeration to state that the floods of last winter carried down enough fertilizing material to produce millions of bushels of wheat, could it have been placed at the disposal of the tiller of the soil.

In Egypt these floods of red water would have been welcomed with feastings and thanksgiving in the mosques, for they would give assurance of bountiful crops. If engineers would study Egypt and follow the teachings of her long experience, in so far as conditions admit, they would be trying no experiment. Willcocks, in his great work, "Egyptian Irrigation," says:

"One cannot study the principles of basin irrigation without admir-

ing the skill and order of the whole operation. However much fault may be found with the unskilful treatment of the new summer irrigation of Lower Egypt, the basin irrigation of Upper Egypt, gradually developed through 5 000 years, commands sincere admiration."

Basin irrigation is dividing the land with dikes into so-called basins and introducing flood-water, usually carrying considerable sediment, from 2 to 6 ft. deep over the entire area, and letting it stand for several weeks until the sediment has settled and the water has soaked into the soil as much as it will. The water is then drained off quickly, and the crop is sown on the mud, often before it is dried sufficiently even to harrow. The areas of these basins depend largely on the slope of the land. In Egypt they vary from a few acres to, in one instance, 40 000, the average being about 5 000 acres. This system of irrigation would be especially beneficial to alkali lands. The experience in Egypt is that where land has deteriorated and shows white efflorescence from perennial irrigation, one or two years of basin flooding restores it to its former state, and in no case has basin irrigation produced alkali lands. So marked is this result that Willcocks intimates that Lower Egypt may yet have to return to basin irrigation, and to wheat raising instead of cotton, tobacco, and sugar, which are much more profitable but require perennial irrigation. Since that time a compromise method of flooding a portion of the land every year with red water has been brought about by raising the Nile at the barrage so as to deliver the red water on the Delta lands, thereby fulfilling Willcocks' famous saying, that the work of the English engineers was to deposit the sediment on the lands and not in the canals. It will be seen, therefore, that in those districts where perennial irrigation has raised the land-water level and brought on the alkali, the basin system will dovetail in and enable the tiller of the soil to wash away his alkali whenever troublesome, and, at the same time, probably lower the land-water level by its quick drainage, as one of the fundamental elements of the basin system is ample drainage channels from every part of the watered area.

No one appears to have realized the great physical and climatic similarity between Egypt and the Great Valley, nor how perfectly the basin system of Egypt adapts itself to and solves the problem in California. Except where perennial irrigation obtains, both countries are limited by the climate to one short-season crop, generally

some food grain, and, as far as can now be judged, this limitation will always hold.

The Egyptians welcome the muddy flood, and gently spread it out over the valley under perfect control, keeping it until it has watered and enriched the land, and then sending it quietly to the sea. Sowing their grain upon the ooze ere the water has barely left it, their crop comes to maturity without other aid. In this way the flood can be mastered and controlled and made to spread peacefully over the floor of the Great Valley, where it will water and enrich the land, and then pass on to the sea, leaving the certainty of full crops behind it. Contrast this with placing dependence on a precarious rainfall to grow an inferior crop on a deteriorated soil, and unsuccessfully fending off the flood in terror lest it destroy the country.

There is one important difference, however, between the Nile and the Great Valley. In the former, the river is expected to fall in time to drain the water from the basins back into it, thus it is both the feeding and draining channel for the basins. Sometimes the expected does not occur, and loss of crops follows. In California the rivers do not usually fall sufficiently to drain the adjoining lands in time for a crop to be planted in the same season, and this may be the reason why the basin system has not been adopted. Other drainage or escape channels must be provided which will drain the basins early in the spring. While these channels will prove expensive, there is ample compensation for them in the fact that the rivers, being high late in the season, can furnish a second irrigation for the wheat in June, which will be of vast benefit to the maturing grain.

It is proposed, therefore, to construct dikes to form basins, as in Egypt, over the entire floor of the Great Valley, comprising some 3 000 000 acres; and to feed these basins, during the winter or flood time, by suitable regulating gates and dams, from the various rivers and creeks entering the Valley; and to regulate and control the feeding of the basins so as to relieve the rivers of flood waters, as much as possible, and hold these flood waters in the basins, or let them move slowly through, that they may deposit the silt and soak the land, and finally drain through escape channels in time for the crops to be sown in the spring. It is proposed to provide escape channels, through the lowest parts of the Valley, of sufficient capacity to drain the basins rapidly, if required, and assist the rivers in times of excessive floods.

In the Sacramento and San Joaquin Rivers, it is proposed to construct, in connection with the basin-feeding regulators, movable dams with locks, which will give slack-water navigation at low stages of water and offer no obstructions during floods. It is also proposed to construct dams or barriers in the rivers, at the margin of the Valley, which will prevent the movement of mining débris or other material now in the rivers, and to raise these barriers as needed, to prevent the future wash from the hills from flowing into the navigable channels.

It is impossible, of course, in this paper to lay out in accurate detail a plan of basin irrigation and its auxiliary works, for the Great Valley, but it is thought that sufficient may be shown to prove the feasibility of the redemption of the Valley by the means which have been in use for centuries in the Valley of the Nile. The success there proves, before work is begun, that it would not be an experiment.

On the map of the Valley, Plate LVII, which is reduced from the remarkably graphic map of 1887, by William Ham. Hall, M. Am. Soc. C. E., the basin dikes in the Sacramento Valley are indicated in heavy dotted lines as a diagram only, and are not supposed to be accurate. In fact, they are simply interpolated 5-ft. contours. They are not indicated in the San Joaquin Valley, because in that valley there are so many complications with perennial canals and other conditions.

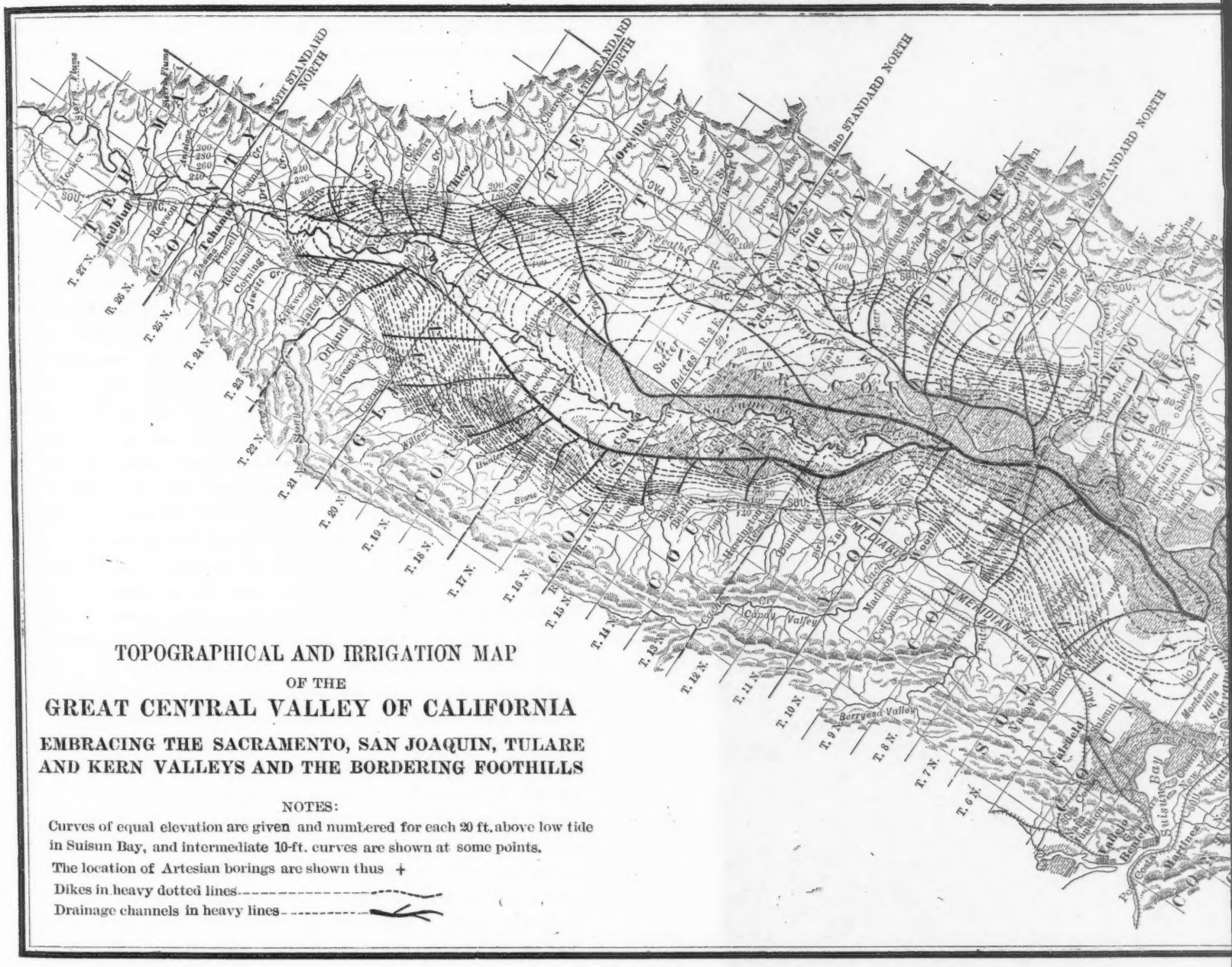
The proper location of the dikes will require not a little care and skill—nearly as much as in locating a railroad line. The surveys and maps now being made by the Reclamation Service are the first necessity. Nearly all the construction can be mechanical. Some dredging or ditching machine of the Aurora type can be especially adapted and built for the service, as there is enough work to be done to pay for special machines. The shape and size of the basins depend chiefly on the slope of the land, for when they are filled there should be a depth of not less than 2 ft. of water anywhere, otherwise the shallower portions will not get their due share of silt.

The dike proposed is 9 ft. high, 8 ft. wide on top, with side slopes of 2 to 1, making a bottom width of 44 ft. This gives 4½ ft. as the average depth of water, with 2 ft. against the upper and 7 ft. against the lower bank. Half the earth would be taken from each side of the embankment, thus making an excavation or ditch which will be of great value in the distribution of the water, both in filling and emptying the basin. An ordinary ditching plow can be used to cut the origi-

nal surface on the center line of the dike, thus making a deep break and a fresh bond with the filling. It is worth while, especially in adobe land, if water can be procured, to settle or puddle the dikes as rapidly as they are built, otherwise they will not hold up much water until after they have stood through at least one rainy season. The dike should be built with the machines, nearly complete, with a trench along the center of the top, and then, at proper intervals alongside the dike, small driven-pipe wells should be put in. With a portable power pump, taken along the line, water can be pumped into the trench and will seep through the entire dike and cause it to settle into a solid mass, care being taken not to water too much nor too fast, thus making mud and causing the bank to slump. There should be drain pipes at intervals along the trench to carry off the heavy winter rains, as, at that time, no water should stand on the dikes. Willow cuttings stuck into the bank will grow rapidly during the first season if kept watered from the trench. These willows are very effective against wave wash, and should be cut every year after the flooding and kept watered through the dry season by pumping into the trench. This may seem to be a trivial matter and a needless expense, but it is of great importance to the safety and permanency of the dikes, and keeps them from being washed by the rains as well as the waves, keeps the earth in place on the slopes, discourages gophers, and tends to a more thorough inspection of the dikes.

It is desirable to feed each basin with water directly from the supply, in order to get all the sediment, and also drain directly to the escape channel, to have it act quickly and thoroughly. In actual practice in Egypt there are many adaptations and complications of this simple proposition, compromising to fit the natural conditions of each locality. There may be a system of basins, each opening to the next, the supply being from the upper to the lower and the draining through the lower from the upper, while the escape channel is by a siphon under the supply channel of another and lower system of basins.

In the Great Valley, the slope of the land is such that most of the basins are very long and narrow, as for instance the 35-ft. contour dike, shown on Plate LVII as starting from near Cache Slough and reaching up the west side of the valley to the Sacramento at Winn's Landing. Both for safety and convenience, this is cut into many basins by cross-dikes, some of them fed from the Sacramento and others from the


TOPOGRAPHICAL AND IRRIGATION MAP
OF THE
GREAT CENTRAL VALLEY OF CALIFORNIA
EMBRACING THE SACRAMENTO, SAN JOAQUIN, TULARE AND KERN VALLEYS AND THE BORDERING FOOTHILLS
NOTES:
Curves of equal elevation are given and numbered for each 20 ft. above low tide in Suisun Bay, and intermediate 10-ft. curves are shown at some points.
The location of Artesian borings are shown thus +
Dikes in heavy dotted lines
Drainage channels in heavy lines
SHASTA COUNTY
TEHAMA COUNTY
GLENN COUNTY
COLUSA COUNTY
YUBA COUNTY
BUTTE COUNTY
PLACER COUNTY
SACRAMENTO COUNTY
YOLO COUNTY
SOLANO COUNTY
MT. DIABLO MERIDIAN
Sacramento River
Feather River
Sutter Buttes
Cache Creek
Clear Lake
Putah Creek
Cosumnes River
Suisun Bay
Berryessa Valley
Capay Valley
Napa Valley
Redding
Red Bluff
Orland
Willows
Colusa
Marysville
Oroville
Chico
Woodland
Davis
Vacaville
Fairfield
Benicia
Martinez
Sacramento
Folsom
Auburn
Roseville
Lincoln
Rocklin
Dixon
Vallejo
Napa
Suisun
Rio Vista
Isleton
Courtland
Walnut Grove
Galt
Elk Grove
Florin
Antioch
Pittsburg
Concord
Walnut Creek
Danville
San Ramon
Livermore
Tracy
Stockton
Lodi
Manteca
Ripon
Modesto
Turlock
Merced
Los Banos
Madera
Fresno
Kingsburg
Selma
Hanford
Visalia
Tulare
Lemoore
Corcoran
Kings River
Tulare Lake
Kern River
Bakersfield
Buena Vista Lake
Kern Lake
Tehachapi Mts.
Coast Range
Sierra Nevada
Mt. Shasta
Mt. Lassen
Mt. Diablo
Monterey Bay
San Francisco Bay
Golden Gate
Pacific Ocean
Mt. Hamilton
Mt. Tamalpais
Mt. Helena
Mt. Konocti
Mt. Vaca
Mt. Oso
Mt. Pinos
Mt. Whitney
Mt. Tyndall
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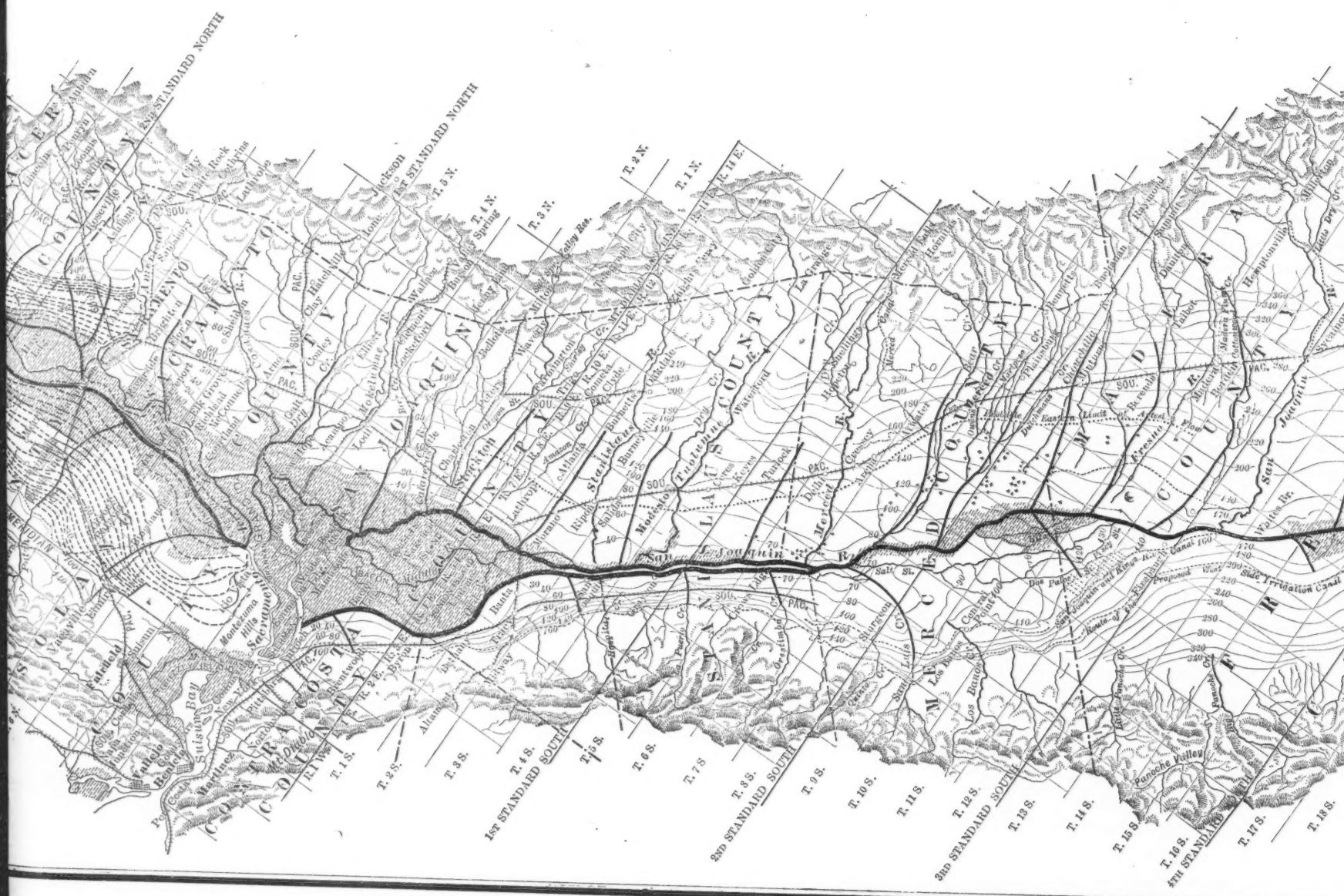
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Stanislaus R.
Tuolumne R.
Merced R.
Fresno R.
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Salt Sl.
Los Banos
Dos Palos
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Mendota
Sta Rita
Hollister
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Livermore
Pleasanton
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Benicia
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Cajon Pass
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Antelope Valley
Mojave Desert
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Owens R.
Kern Lake
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Elk Hills
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Oil City
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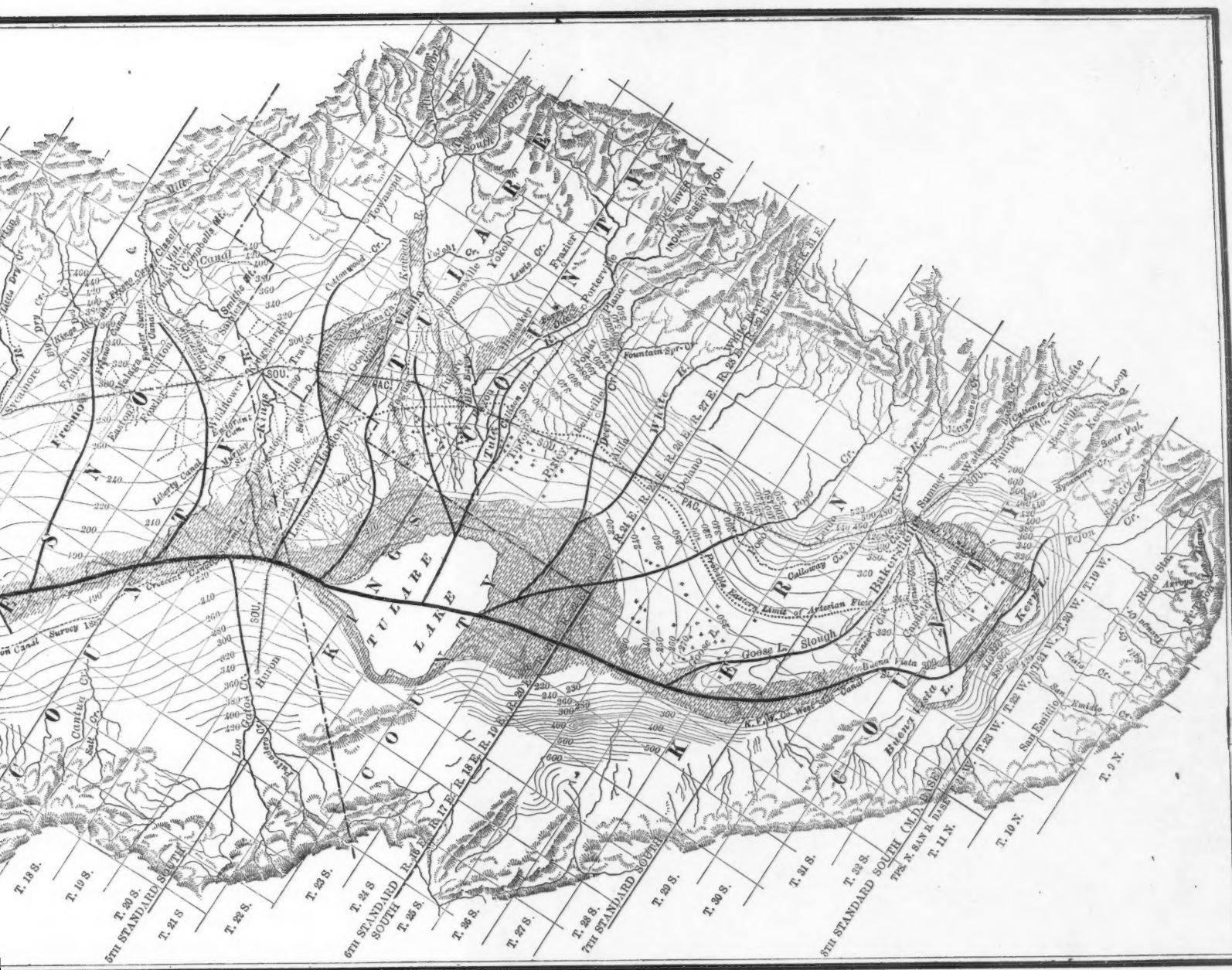
PLATE LVII.
PAPERS, AM. SOC. C. E.
SEPTEMBER, 1909.
FOOTE ON
REDEMPTION OF GREAT VALLEY OF CALIFORNIA.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Sutter
Dinuba
Visalia
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Farmersville
Woodlake
Ivanhoe
Lemoon
Traver

side streams, Putah Creek, Cache Creek, etc. When completed, therefore, the basins may be from 10 to 20 miles long and from 1 to 3 miles wide, forming a manageable shape and area.

The feeding regulators, dams, and gates will try the skill of the engineer, especially along the slopes where side streams are to be taken care of. They must be built, not only to feed the basins, but to allow any excess to pass, and they must be built on sand. Much may be learned from the experience of English engineers in India about hydraulic structures on sand foundations. They have done a vast amount of monumental work of this kind, under conditions far more difficult than any in the Great Valley. They have put their head-works and anicuts, as they call them, across sandy-bottomed streams, and these have withstood floods of more than twice the volume ever given by the Sacramento and San Joaquin Rivers combined.

Recent practice in the use of cement grout in sand and gravel foundations will enable engineers to accomplish results in these sandy river bottoms at far less cost than the Indian structures of stone and brick; and the great factories near Vallejo will furnish this most important article, cement, at a price which will allow of its liberal use. Few people know and few engineers seem to realize the value of cement grout for sand and gravel foundations. The cantilever bridge across the Niagara Gorge is built on the loose slide or talus which fell from the bluffs. A place on the slope was leveled off and cement grout was poured in among the loose stones until the interstices were filled; this, on hardening, gave a solid mass on which to build. A cement conglomerate it might be called. The great barrage at the head of the Delta of the Nile, built on Nile mud, was nearly useless for many years until Scott-Moncrieff strengthened the foundations with broken stone filled with cement grout.

The sand and gravel beds of the streams entering the Great Valley are perfect material for forming this cement conglomerate if cement grout is forced into it. The operation is quite simple and rapid. Pipe is driven into the river bed, if possible by the water-jet method, to the depth required for the foundation, and a force pump drives the grout into the interstices until they are full of cement which on hardening forms the solid concrete. The grout may be forced in within a radius of from 3 to 6 ft. from the pipe, depending on the porosity of the bed material. In this manner, broad and deep foundations may be con-

structed on which to build movable dams and regulators, on nearly every stream entering the Valley, at a cost of less than half the usual estimates for such work. Near the center of the Valley, where clay or adobe is encountered, this method cannot be used.

One or two of these structures for regulation will be required on each of the smaller streams, and several on some of the larger ones. Each will have to be designed to fit its own peculiar conditions, after those conditions have been determined by careful detailed surveys. In the navigable streams it is proposed that the feed-regulating structures shall include a movable dam and navigation lock for slack-water navigation. These are necessary in order to turn all the flowing water into the basins, if required, during a dry season, or to open out and afford

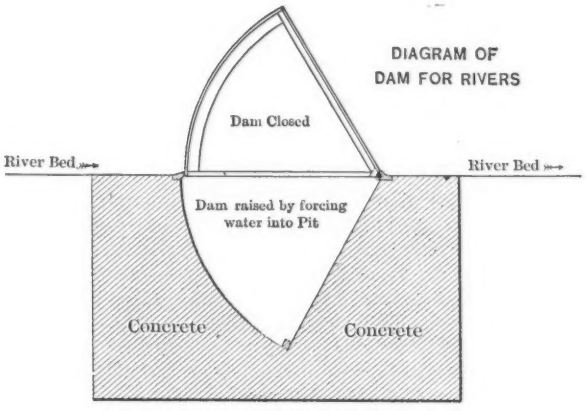

FIG. 1.

no obstruction during a heavy flood season, and also to allow all the water to be used for perennial irrigation after the snow flood in June. For these movable dams and also for the feeder gates it is proposed to use a construction similar to the sector dam or gate designed by Mr. E. R. Cooley and used by Isham Randolph, M. Am. Soc. C. E., at the locks and power plant of the Chicago Drainage Canal. As there is no ice to interfere with them, the only apparent objection to them is removed. If worked by hydraulic power (obtained by pumps, in this case), as at Lockport, it would appear feasible to have a single gate for the entire width of the river, or perhaps two with one pier between them, which, when down, would give a very free waterway for floods. It might be preferable to have the feeder gates in comparatively

short sections, and arrange them with the axis on piers and the gate to be raised out of the water by suitable gear. A counterweight might be added by extending the radials of the structure, as was done by A. J. Wiley, M. Am. Soc. C. E., for canal gates, which makes the manipulation very easy. Figs. 1 and 2 indicate the design of the gates in a general way.

The proposed escape channels are indicated by heavy lines on the map. They are drawn very roughly and tentatively, as they are of the first importance and their location and size can only be determined by hard and skilful work. In the Sacramento Valley, it is proposed to use the Cache Slough, widened to the proper dimensions. With a bottom grade beginning at mean low water, a wide channel should be

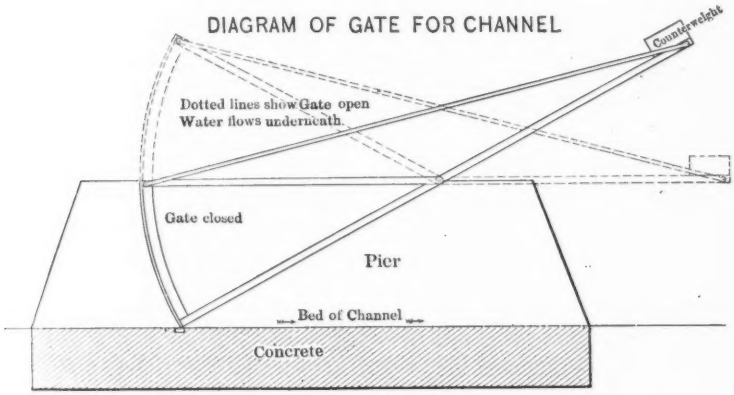

FIG. 2.

carried up the lowest part of the Yolo Basin, with a grade, if possible, which will give from 4 to 6 ft. of natural banks. If feasible, it would be better to use the excavated material for filling or leveling up low places, and have no spoil banks, the design being to use the whole area between the 10-ft. contour dikes as an escape channel when desired, and to use the excavated channel permanently for ordinary drainage and for carrying the excess water of Cache and the other west-side creeks. There would always be a very considerable seepage gathered by the branch channels which would have to be taken away by this main channel.

Above Sacramento a branch channel must be carried under and across the river to drain the American Basin, and should extend north-

ward toward Marysville. There seems to be enough fall to allow of an appreciable head for the siphon under the river, though even then it will need a large cross-sectional area and will be an expensive undertaking. Another branch escape channel will have to be carried under the Sacramento, to take the water from the Sutter and Butte Basins. Adopting for the river the cut-off proposed by the Commission of Engineers, the siphon could be placed in that, while building, and the old channel of the river could be used and extended northward through the Colusa Basin.

The present channels on the west side of the valley, and those of the smaller streams on the east side, can be used both as escape and feeder channels, as their flood season will be over in time to receive the escape waters in the same channel.

The siphons under the Sacramento may seem to be serious obstacles to the system, but on due consideration the objections to them may be reduced to the simple one of cost. There is nothing novel about them except their size, and, if considered as structures only, there is no novelty in their size. The Solani or Nadrai Aqueducts, which pass the Ganges Canal over streams, are far larger and more expensive. Many smaller siphons are in use in Egypt. There are three large siphons in the Cavour Canal, Italy, and as for permanence there is one in Spain, built of masonry by the Moors, which has 300 ft. head and has been in continuous service for more than 600 years, without accident as far as known. If the location should prove to be in sand, the grouting method would reduce the cost materially, and make better foundations, as water would not seep along the grouted portion, and if the upper part were built of concrete the cost would be very low in comparison with the brick and masonry constructions in India. If it is necessary to build these siphons to drain from the basins an average depth of 5 ft. of water in 30 days, it will require a capacity of about 30 000 cu. ft. per sec. for the American Basin and 50 000 cu. ft. per sec. for the Sutter. Assuming that there will be sufficient slope to give a head on the siphon which will force the water through at a velocity of 8 ft. per sec., the siphon for the Sutter Basin would have thirty barrels or archways, each 20 by 10 ft., and eighteen would be required for the American, or the former structure would be about 700 ft. wide and as long as the distance, outside to outside, of the river levees, and the latter would be about 425 ft. wide and of similar length. It is probable

that these siphons and the feeding and regulating dams and locks would be located as one structure in the river, which would reduce the cost appreciably. In any event, it is evident that the cost of the siphons would not be prohibitive, and would be cheaper than any competent pumping plant, even for drainage purposes, if the cost of operation be considered. It should be added that the rectification of the Sacramento River below Cache Slough, as recommended by the Commission of Engineers, is a very necessary part of the escape channel system.

The escape channels for the San Joaquin Valley are also indicated in heavy lines on the map, and have been drawn with considerable hesitation, as there is much less information on record concerning the physical conditions in this valley than in the Sacramento. It is not probable, however, that the San Joaquin River, from the Merced down, can be depended on to subside soon enough to carry off the escape waters from the basins. It is proposed, therefore, to carry an independent channel up on the west side, as shown on the map, to intercept the San Joaquin just above the Merced, and thence, following and enlarging the river, to its bend to the eastward. Thence, it would follow the lowest part of the valley, with a slope sufficient to drain Tulare Lake completely. This would necessitate a long cut from about the 190 contour to the center of the lake, which would be expensive, but it would reclaim the whole area of the lake and its swampy margins. It is probable also that the channel would be continued through the lake and slough to Buena Vista Lake. The San Joaquin River below the Merced will need to be rectified and provided with dams and locks for slack-water navigation, and connected with the upper channel so as to make a navigable channel from the Bay to Lake Tulare. Branch escape channels for draining the basins will be needed, somewhat as indicated on the map.

In the matter of barriers for holding back the mining débris and other wash from the mountains, it may be thought that the State's attempts in the Yuba tend to discourage any more work in that direction. The work done, however, should be remembered rather as a lesson well learned, to be used to future advantage. It appears from the reports that in each instance of the breaching of these barriers it was caused by undermining. That is, the water, after passing over the barrier, dug a hole on the lower side of it and worked back underneath until the barrier fell from want of support. A hint may be

taken from the English engineers in India; for a wall may be carried down into the river bed on the lower side of the barrier far enough to prevent this undermining. This would remedy the immediate and local danger to the barriers, but there is a general action going on which must be remedied also. It is so well known as to need no comment, that rivers if left to themselves are eternally flattening their grade. That is, they are moving whatever forms their bottoms along down stream in the usual course of Nature, and the steeper the grade the greater the action. If, therefore, some extraordinary cause fills up a river and makes its bed much steeper, this cutting action will become rapid, and if, in addition to this, at a certain point in the river, all material coming from above be stopped, and held, then the cutting action below this point will be much more rapid, because, as the material is moved along, there is no other from above to take its place.

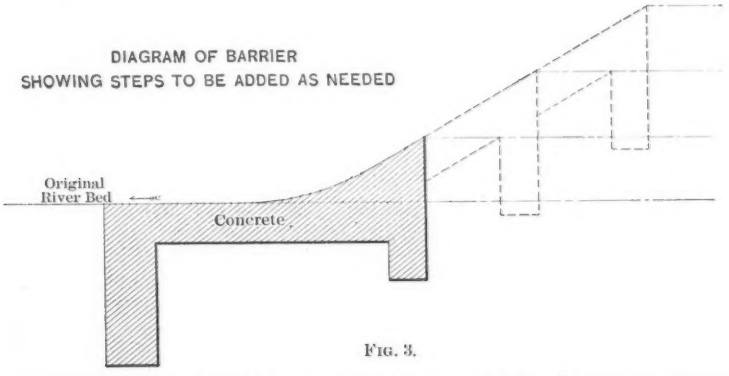

FIG. 3.

If barriers are placed in the Feather and Yuba Rivers up near the hills only, the movement of the vast accumulation of débris below the barriers will simply be accelerated, and they will pass downward into the Sacramento and also soon undermine the barriers. As this material should not be moved into the Sacramento, and as it is desirable to have barriers in order to prevent any more débris coming from the hills, they must be erected at intervals all the way from the Sacramento to the upper barriers; this will break the river grade into steps, with reaches between so nearly flat as to prevent the movement of the coarse débris forming their bottoms. This would lessen the velocity of the current and require higher levees to hold the floods. The excess of the floods must be turned into the basins and pass off through the escape channels.

In building these barriers, shown in Fig. 3, it is proposed to use the cement grouting method for a large portion of the structure. The upper barrier may be added to, as needed, for generations to come, without endangering its stability, and, if the surface becomes worn appreciably, it will be neither difficult nor costly to spread a few feet of concrete over it, during the dry season, and this, if kept moist until it has set, will be as good as the original surface.

As soon as the side streams entering the Valley are placed under control, it is believed that the heavier silt following along the bottom of these streams may be segregated and conveyed in small flumes to reclaim much waste land which is now useless, such as the beds of old sloughs, low places that have become alkaline, and small areas of swamps which cannot easily be drained. In Italy and the South of France, large areas have been reclaimed by this method, or colmatage, as it is called, and much may be learned from their experience; but, in segregating this heavier silt, something may be learned from the intolerable placer miner, who, when he puts in his "undercurrents," as he calls them (for an entirely different purpose), points the way whereby the lower strata or heaviest silt, rolling along the bottom of a channel, may be diverted and carried away. The sketch, Fig. 4, shows the process better than it can be described. The secret of it is that the slot or exit orifice extends entirely across the bottom of the channel, and the velocity of the water through this orifice is not retarded by the sidewise discharge channel underneath. By a suitable device to vary the width of the exit orifice, any desired quantity of the heavier silt may be removed. The only mishap it is subject to in working is that it may get clogged by water-logged sticks and leaves rolling along the bottom with the silt. Opening the orifice wide for an instant will generally clear it. It is thought that a trap of this kind may be located in the heavier silt-bearing streams (they might be built into the flat portion of a barrier) where sufficient grade can be found, and convey the semi-liquid silt in small flumes to localities where needed.

DIAGRAM SHOWING OPERATION OF SILT OR SAND GATE

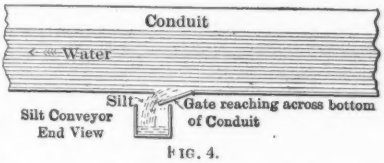

FIG. 4.

Finally, in working for the benefit of the tiller of the soil, the placer miner is reached. With the barriers in the streams to hold the

coarser part of the miners' débris, the finer silt goes to the soil, and the farmer gets more gold out of it, through his crop, than the miner in his riffles. It can be demonstrated in hundreds of instances among the foot-hills, that enormous crops are produced on soil made entirely from the "slickens" of the mines, and even the raw slimes from the stamp mills, collected in terraces in shallow gulches by little brush dams. It seems as if the raw rock, so finely ground, presented its potash and phosphorus to vegetation in a form that can be readily assimilated.

It is not probable that placer mining will ever become as large an industry as it was before it was stopped by law, but any placer mining will benefit the farmer, at the same time it will throw more or less coarse material into the stream beds, and this must be retained by raising the barriers oftener than would otherwise be required. It would be but reasonable, therefore, that the placer miner be taxed sufficiently to pay for the increased work required at the barriers, even if his silt does enrich the valley lands.

It is expected that the Sacramento River will rapidly deepen itself after the barriers are in place, yet there will be much dredging to be done, as shown in the Report of the Commission of Engineers; but, having the escape channels, as provided in this basin system, the river will never be required to carry much more than two-thirds of the greatest flood, and therefore it will only be necessary to enlarge the carrying capacity of the river to about two-thirds of that demanded by the Commission of Engineers, except below Cache Slough. From Cache Slough to the Bay, room must be provided for more than 300 000 cu. ft. per sec. for the Sacramento River, in addition to that required for the San Joaquin below the junction.

Assume the heaviest flood conditions ever known (or those intimated in the first part of this paper), as sure to come in the Sacramento Valley, and imagine the basin scheme complete. It is believed that the operation would be as follows: In the beginning of the flood, the regulating dams would be closed, the feeder gates open, and the basins filling. Ordinary judgment would imply the opening of the escape gates somewhat before the basins were full. As the floods rose, the regulating dams would open, to pass the excess down the rivers. The floods still rising, until all escape gates are wide open, all dams down, and the rivers full, there would still be room in the basins for

considerable excess, until their limit was reached. In this condition, the flood capacity of the river and escape channels would be somewhat more than 100 000 cu. ft. per sec., greater than was ever required in the Valley.

It is reasonable to assume that these conditions would not continue long, and would not occur later than March. Very soon the feeder gates would be closed and the rivers would be carrying the falling flood. The escape channels would drain the basins rapidly. Sowing could begin in the higher basins by the middle of April, and in all by the first of May. This may be thought to be too late for a good wheat crop, but it must be remembered that a heavy flood in the Valley means a heavy snowfall in the mountains and a long snow flood, so that a late sowing of the crops is compensated by a late irrigation, that is, that the feeder gates will be opened again near the end of June, and the crop will then get a heavy irrigation at the time of the hot winds of late June and early July.

From the foregoing it may be seen that a flood 25% greater than that of 1862 could pass through the Valley, not only without injury, but with vast benefit to the crop which would surely follow.

Assume the conditions in an extraordinarily dry season: Unfortunately, knowledge in reference to the weather is not yet sufficient to predict, at the beginning of the winter, what the rainfall is to be, but, in any event, it will always be well to begin turning the muddy water into the higher basins as soon as it comes; and, if there are no rains of consequence following, by closing the regulating dams, all the water there is can be utilized and spread as equably as possible over the basins and give them at least a short soaking. Early sowing will be made, and, as there is always some snow flood, it may be all used for the summer irrigation early in June by closing the dams again. In the extra dry year, therefore, the basin system will utilize all the water there is, and make something of a crop, though the land will not get much silt, and must draw on its previous enrichment to carry its crop.

It is conceded that this scheme, as presented, will cost many millions. On the other hand, it should be conceded that where the cost is millions the return is hundreds of millions.

A reasonable estimate for the average increase in the value of the land benefited is $100 per acre, and there are approximately 3 000 000

acres to come under the system. The increased value of the crop produced annually would be $20 per acre, an annual value of $60 000 000.

The value of the navigable waterways produced by the building of the system is very intimately connected with the value of the land, and may be included in that estimate.

The value of the flood protection cannot well be estimated in dollars, as it includes human life and happiness; but the increased value of real estate in Sacramento and Stockton will be of importance when it is once realized that there is no danger from floods, and that these cities are surrounded by millions of acres of ever rich wheat lands.

The renewal of placer mining, with its demand for supplies, will also contribute to the prosperity of the Valley, and the bitter war between the valley and the hills will end in both being benefited by the "slickens."

It will take a number of years to complete the scheme, and may require $75 000 000, which can be borrowed by the State as needed, and returned to the State by the lands benefited, in installments, after the benefit has been received, in a manner similar to that followed by the U. S. Reclamation Service. The benefits would cover, not only the lands in the basins, but also the cities and towns, and the mines, or all property in the Valley. The assessment might be arranged as a general tax on all property, sufficient to pay the interest; and, as basin lands would receive the most benefit (after they began to produce), a repayment tax of $2.50 per acre for ten years might be imposed, in order to pay back the principal. Afterward, a maintenance tax, sufficient to maintain the system, should be placed on all property.

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PAPERS AND DISCUSSIONS

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THE TRACTION OF FREIGHT TRAINS AT DIFFERENT SPEEDS.*

By Clinton S. Bissell, M. Am. Soc. C. E.

The problem of railroad traction is to determine what a given tractive power can accomplish in overcoming a number of resistances, the principal of which is grade. The "ruling grade" is usually the maximum grade on a tangent, and the increase in resistance which would be introduced by curves on the ruling grade is, in the present day, done away with by equating or "slackening grade" throughout the length of the curve, with the purpose of producing a perfectly uniform resistance over tangents and curves alike. In the case of curves not thus equated, the additional resistance due to the curves must be applied as an increase to the ruling gradient throughout the length of each curve, and, if the proportion of curved line be considerable, the judgment of the engineer must decide upon what the ultimate "resisting gradient" will be.

Curves are commonly equated at rates varying between 0.02% and 0.10% of gradient per degree of curve, and 0.05% is a rate largely used for freight tracks where the speed is slow. It is well known, however, that curve resistance does not vary uniformly with the degree of curve. Among authorities on the subject, the writer finds a general consensus of opinion to the effect that 0.05% for light curves and

* This paper will not be presented at any meeting, but written communications on the subject are invited for publication with it in *Transactions*.

0.02% or 0.03% for very sharp curves is about right, with intermediate values approximately proportional. Using this as a standard, it is apparent that the amount by which the rate of the ruling grade should be reduced on curves is given by the following equation, in which C is the percentage of reduction and D is the degree of curve:

$$C = \frac{D}{D + 18} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

This reduction is for curves on ruling grades where the train is not required to stop. Using Equation 1, a 1% grade becomes 0.9% when equated for a 2° curve, and 0.47% when equated for a 20° curve. In what follows, the term, g, means either the "resisting gradient" or the "ruling grade compensated for curvature," at about this rate.

In a paper* recently presented to the Society, the writer developed the following equation, expressing the form of variation in the case of the weights of heavy freight trains moving at the slow speed of from 10 to 7 miles per hour:

$$\text{Tons of train weight behind tender} = \frac{P - 20\,g\,m}{\frac{a}{b + W} + 20\,g} \quad \ldots\ldots\ldots (2)$$

in which P = the drawbar pull, in pounds, m = the weight of the locomotive and tender, in tons, a and b are constants, W is the average weight per car of the train, in tons, and g is the rate of the ruling grade, in percentage, the ton being of 2 000 lb. For a locomotive, weighing, with its tender, 168 tons, exerting a drawbar pull of 32 100 lb., and hauling a train of modern high-efficiency cars, which is representative of ordinary conditions, this equation becomes:

$$\text{Tons of train weight behind tender} = \frac{32\,100 - 3\,360\,g}{\frac{249.6}{11.2 + W} + 20\,g}.$$

In order to adapt Equation 2 to speeds greater than 10 miles per hour, two changes must be made. As the speed increases, the drawbar pull, P, decreases; and, to a much less extent, the car resistance, $\frac{a}{b + W}$, increases. The variation in the drawbar pull is unquestionably in the form of a curve, and elaborate modern tests† have shown that the rate of change is comparatively low at high speeds, becoming

* "The Maximum Weights of Slow Freight Trains," by C. S. Bissell, *Transactions*, Am. Soc. C. E., Vol. LXIV, p. 303.

† For example, "Locomotive Tests and Exhibits, Pennsylvania Railroad System, Louisiana Purchase Exposition," 1904.

more and more rapid as the speed decreases, until near the limiting maximum value of the tractive power. At this point the curve usually exhibits a quick decline, producing contrary flexure; and, for modern heavy locomotives, this takes place at a speed of between 10 and 6 miles per hour. Such a variation cannot be expressed algebraically without great complication, but fortunately the major part of the curve, above 10 miles per hour, may be expressed very simply as an algebraic equation. From the records of the Pennsylvania Railroad Company the writer obtained the following data pertaining to a heavy freight locomotive of the type used in this study:

Speed, S, in miles per hour.	Drawbar pull, P, in thousands of pounds.
10	32.10
16	22.37
30	10.35

Observing that a suitable form of equation for a curve through these points is:

$$P = \frac{h - k\,S}{n + S},$$

in which h, k, and n are constants, and S is the speed; and deducing the constants, the expression for the drawbar pull becomes:

$$P = \frac{863.5 - 14.1\,S}{12.5 + S},$$

in which P is in thousands of pounds, and decreases as the speed increases, in proper proportion, as shown on Fig. 1.

The second step is to find an expression for the increase of train resistance, in pounds per ton, in proportion to the increase of speed. For this purpose the writer refers to the paper* "Virtual Grades for Freight Trains" by A. C. Dennis, M. Am. Soc. C. E. Fig. 1 of that paper exhibits several curves of train resistance, and, commenting upon them, Mr. Dennis says:

The train resistance, in pounds per ton, compensated for change in velocity head, grade and curvature, was plotted for very many points and speeds for each train, and the mean of these points taken, as nearly as possible, for each train. Fig. 1 shows what is believed to be the correct resistance under the conditions given, and differs radically from the results obtained by Wellington and other authorities experimenting with light trains, cars and rails."

* *Transactions*, Am. Soc. C. E., Vol. L, p. 1.

For this investigation the writer selected the curve marked "52 Loaded Box Cars. Tare 37% of Gross Weight. Track Soft," and used only that portion which is for a speed greater than 10 miles per hour. Taking the ordinates by scale, the following was obtained:

Speed, S, in miles per hour.	Resistance, Y, in pounds per ton.
15	4.50
25	4.73
35	5.00

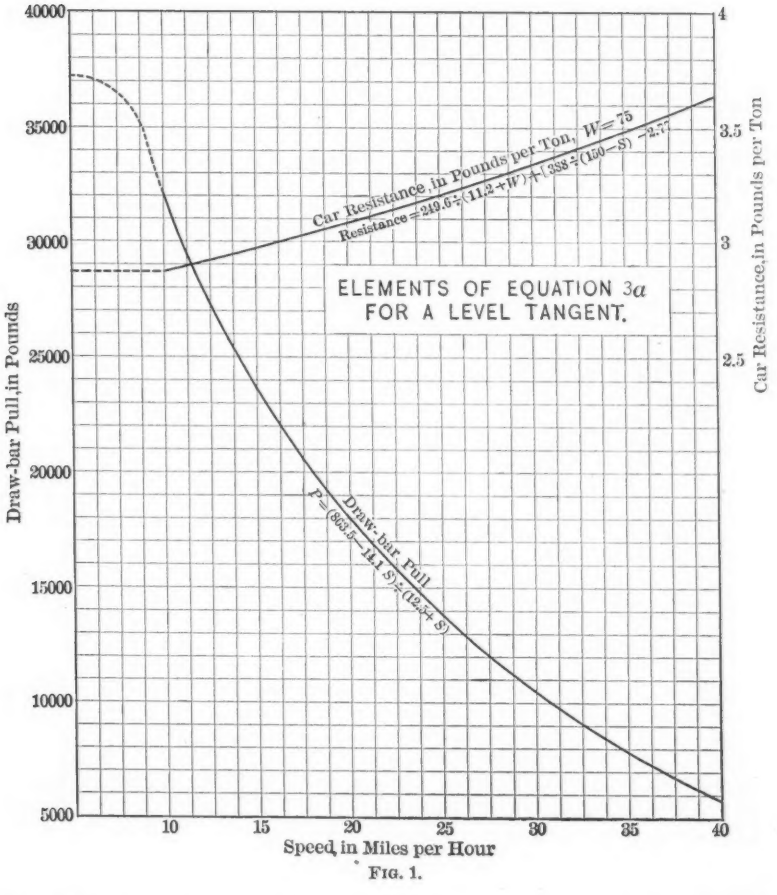

FIG. 1.

in which the resistance, 4.73, is about 0.1 greater than as shown by the curve. In the light of Mr. Dennis' comments, this slight modification will not vitiate the accuracy of the following expression for resistance,

which was deduced in the same manner as that for drawbar pull. The curve,

$$Y = \frac{388}{150 - S} + 1.625,$$

passes through the three points tabulated above. Had the modification not been made, the numerator of the fraction would have been smaller and the denominator, $150 - S$, would have become about $47 - S$. For speeds near 47 the variation would then have become much too great, since the curve has a rapid flexure on approaching its asymptote, $S = 47$. Under the circumstances, this slight change seems preferable to the complication necessary for a more exact treatment of the case.

The value of Y for a speed of 10 miles per hour, according to this equation, is 4.396 lb., and the increase in resistance at this speed must be zero, hence the constant, +1.625, becomes —2.77, and the expression for increase in resistance due to speeds exceeding 10 miles per hour is:

$$\text{Increase in car resistance} = \frac{388}{150 - S} - 2.77.$$

Denoting the constants by the letters, r, t, and c, and substituting the two expressions in Equation 2, the general equation is formed:

$$\text{Train weight, in tons, behind tender} = \frac{\dfrac{1\,000\,(h - k\,S)}{n + S} - 20\,g\,m}{\dfrac{a}{b + W} + \dfrac{r}{t - S} - c + 20\,g} \quad \ldots (3)$$

in which S is the speed, in miles per hour, W is the unit of train weight behind the tender, expressed as the average weight per car, in tons, and the remaining letters are constants, as heretofore designated. Substituting in Equation 3 the constants for the locomotive and cars used in the given example, there results the equation:

$$\text{Train weight, in tons, behind tender} = \frac{\dfrac{1\,000\,(863.5 - 14.1\,S)}{12.5 + S} - 3\,360\,g}{\dfrac{249.6}{11.2 + W} + \left(\dfrac{388}{150 - S} - 2.77\right) + 20\,g} \quad \ldots (3a)$$

It will be noted here that the train weight varies with a multiple of the square of the speed, which is in accordance with past experience.

As a matter of suggestion, the writer wishes to call attention again to the expression for increase in train resistance due to speed. Regarding the curve passing through the points, $S = 15$, $Y = 4.5$, and $S = 35$, $Y = 5.0$, as a circle tangent to the direction of abscissas at $S = 0$, and applying the principle that "the square of any distance

divided by twice the radius will equal the distance from tangent to curve very nearly"* the radius is found to be 1 000. This gives the rate of increase in resistance as 0.0005 S^2, increasing from $S = 0$. For $S = 10$ the value is 0.05, and therefore the increase in resistance

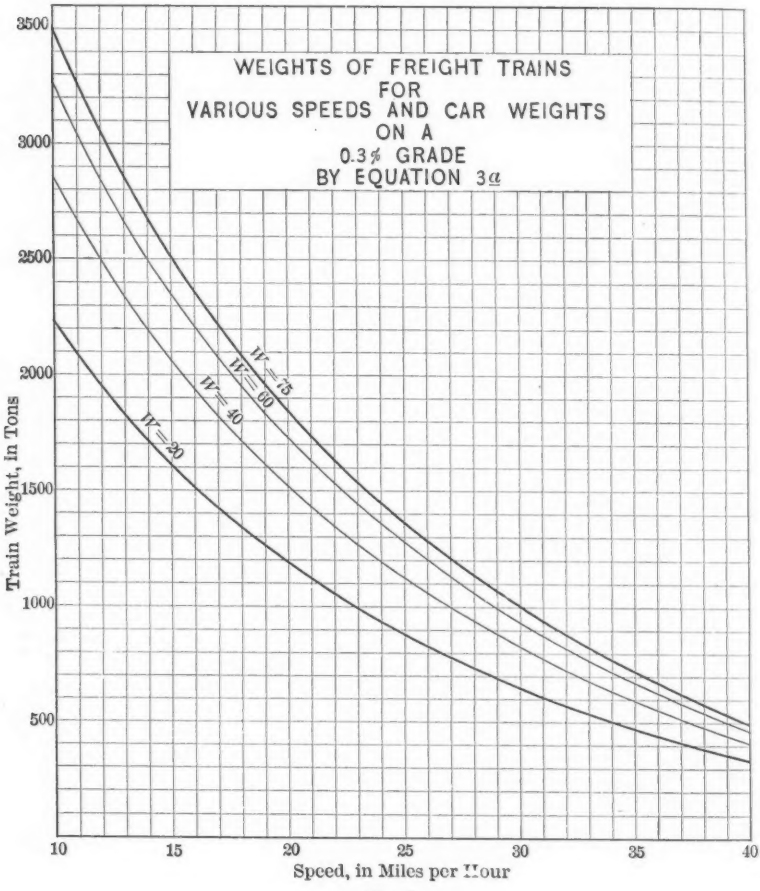

FIG. 2.

above 10 miles per hour will be (0.0005 S^2 — 0.05), which may be substituted in Equation 3*a* for the expression $\left(\frac{388}{150 - S} - 2.77\right)$, if desired. The results are practically identical, the difference at 50 miles per hour being only 0.09 lb. per ton. The writer prefers the

*This statement is to be found appended to tables designed by George T. Keith, M. Am. Soc. C. E.

expression containing only the first power of S, because of its greater simplicity, and its flexibility in adaptation to data obtained by experimental tests. The curve on Fig. 1 marked "Car Resistance" shows the form of variation for an average car weight of 75 tons.

The results of Equation 3*a* for car weights of 20, 40, 60, and 75 tons, on a 0.3% grade, are exhibited in Fig. 2; and for similar car weights on a 1% grade in Fig. 3. The great effect which the car weight has is apparent from these curves.

The writer had completed the form of Equation 3*a* before he realized that the expression for car resistance was a form over which there has been much discussion and disagreement in years past. Those interested will find in "Economics of Railroad Construction," by Walter Loring Webb, M. Am. Soc. C. E., a carefully drawn comparison of formulas for "train resistance" identified with the names of Baldwin's Locomotive Works, Wellington, Barnes, Aspinwall, and Searles. All these formulas are of the form:

Resistance, in pounds per ton = constant + multiple of velocity.

Professor Webb calls attention to the fact that only Searles' formula and Wellington's (which was based on Searles') contain a factor varying with the train weight, and he emphasizes the necessity of this element in any expression which aims to comprehend the entire range of variation. It is noticeable that the constant in the above form was evidently chosen to represent an average value for the expression, $\frac{a}{b + W}$, in Equation 3, and that, in the two formulas just mentioned, a part of the resistance is made to vary with the total weight of the train, thus rendering them more accurate for extremes of velocity.

It will be observed here that the factor, W, the average weight per car, is representative and comprehensive of all such terms as "weight the train," multiples of "length of train," "number of cars," or similar factors.

The remarks* of the late A. M. Wellington, M. Am. Soc. C. E., whose formulas are mentioned above, indicate a doubt as to his own determination of the proper total value of the various resistances. As a heading for some tabulated values he writes:

"Table 166. Train Resistance on a Level as Affected by Velocity Giving what may be considered as the ordinary working maximum, as

* "The Economic Theory of Railway Location," by A. M. Wellington, M. Am. Soc. C. E.

computed from the general formula of Wm. H. Searles, coinciding closely with the apparent indications of the most recent* tests, but possibly as much as *one-third too high* for the resistances at high speeds under favorable conditions * * *."

The principal resistances here referred to are evidently those of the atmosphere. To the writer's mind, the carefully executed tests made by Mr. Dennis have demonstrated that normal atmospheric resistances

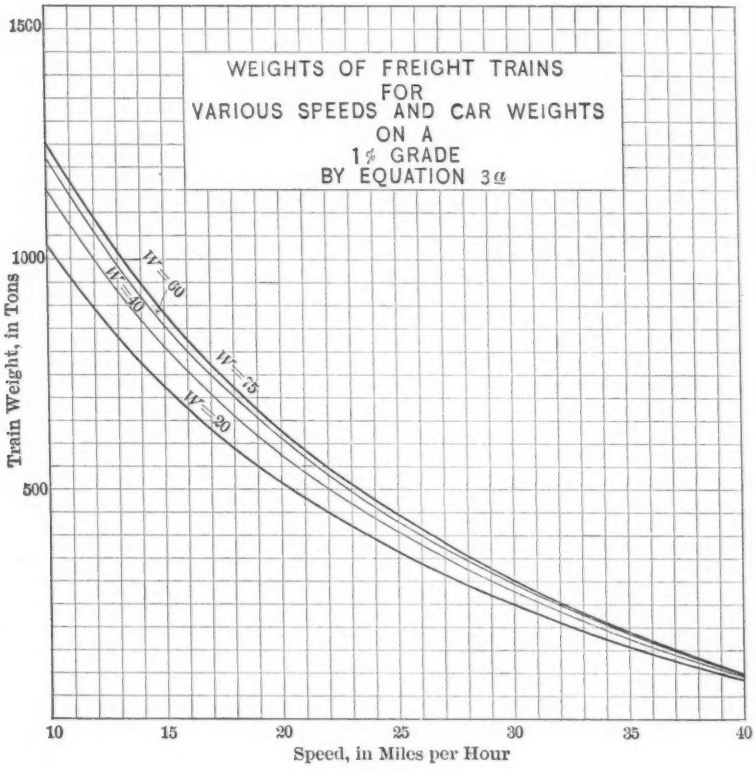

FIG. 3.

are not nearly as great as they were formerly believed to be; and, in using these tests for the development of Equation 3*a*, the writer believes that the atmospheric resistances are represented by the slight upward trend of the curve used to show this variation.

Professor Searles says:† "A formula which shall express the resist-

* Not later than 1887. (The writer.)

† "Field Engineering," by William H. Searles, M. Am. Soc. C. E.

ance of a train to uniform motion must include at least the velocity and the weight of the train and engine."

In the development of Equation 3*a*, the writer has regarded the engine as merely the extraneous agent which supplies the drawbar pull. Since the drawbar pull contains no element of the internal or other resistances of the engine, the form of development has made it unnecessary that the engine weight should appear in the equation.

It has been difficult to obtain data with which to compare the results given by Equation 3*a*, but such comparisons as could be made have been satisfactory. In formulating the equation, the great object has been to deduce expressions which would represent the variation accurately to a point well beyond the extreme limit of speed. Recognizing that the problem is not one of absolute precision, the writer has in all cases used curves involving only the first power of the independent variable in preference to more complicated forms. The constants for each of the three curves, that is, the drawbar pull, the car resistance, and the increase in car resistance, must be determined by actual tests, and, therefore, upon the excellence of the tests depends the accuracy of the results. From good tests the constants can be easily deduced for each curve by a simple algebraic process. When this has been done, the car resistance constants may be retained, and only those in the expression for drawbar pull will need to be changed in order to suit the type of locomotive used.

The great disparity between the results of formulas which have heretofore been deduced to express the resistance of a train at different speeds seems to have arisen from either a failure to recognize the effect of the relation of the train weight to the number of cars, or else from an attempt to deduce an average value to represent it, which is obviously impossible. The absolutely correct formula would require an expression for the speed and also one for the weight and resistance of each particular car, which is impracticable. The nearest approach to this form would be an equation containing the speed factor, the number of cars, and the weight of the train, and this is the form of Equation 3, except that the two latter factors have been merged into one, "the average weight per car" of the train, which is a most convenient expression for the purpose to be served.

Equation 3 is adapted for use in all cases of hauls made by locomotives of either steam, electric, or other motive power; and also in

cases of multiple-unit trains since each unit may be considered as a separate train. The general method of developing the three curves will be the same as exemplified above, except that the form of one or more of them might need to be changed to suit special forms of cars. It does not appear probable, however, that such a change will be necessary, for many years at least, except perhaps in the curve for the increase in car resistance due to increase in speed. It is with great interest that the writer presents this development, in the hope that the discussion may bring out whatever fallacies or errors exist, in order that they may be rectified, if possible.

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PAPERS AND DISCUSSIONS

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THE NEW YORK TUNNEL EXTENSION OF THE PENNSYLVANIA RAILROAD.*

By Charles W. Raymond, M. Am. Soc. C. E.†

Some time before the appointment of the Board of Engineers which supervised the designing and construction of the New York Tunnel Extension of the Pennsylvania Railroad, the late A. J. Cassatt, then President of the Company, said to the writer that for many years he had been unable to reconcile himself to the idea that a railroad system like the Pennsylvania should be prevented from entering the most important and populous city in the country by a river less than one mile wide. The result of this thought was the tunnel extension project now nearly completed; but it is only in recent years that new conditions have rendered such a solution of the problem practicable as well as desirable.

Previously a tunnel designed for steam railroad traffic, to enter New York City near Christopher Street, was partly constructed, but the work was abandoned for financial reasons. Then plans for a great suspension bridge, to enable all the railroads reaching the west shore

* This paper and that of Mr. Noble are printed as introductions to others describing the New York Tunnels and Terminal of the Pennsylvania Railroad, which will appear in the October, 1909, and subsequent numbers of *Proceedings*.

† Brigadier-General, U. S. Army, *Retired*; Chairman, Board of Engineers, Pennsylvania Tunnel and Terminal R. R. Co.

of the North River to enter the city at the foot of 23d Street, were carefully worked out by the North River Bridge Company. The Pennsylvania Railroad Company gave this project its support by agreeing to pay its *pro rata* share for the use of the bridge; but the other railroads declined to participate, and the execution of this plan was not undertaken.

New operating conditions, resulting from the application of electric traction to the movement of heavy railroad trains, which had been used initially in tunnels by the Baltimore and Ohio Railroad and was subsequently studied and adopted by railroads in Europe, made it possible to avoid the difficulty of ventilation connected with steam traction in tunnels, and permitted the use of grades practically prohibitive with the steam locomotive. The practicability of the tunnel extension project finally adopted was thus assured.

The acquisition of the control of the Long Island Railroad by the Pennsylvania Railroad Company, which occurred in 1900, introduced new and important elements into the transportation problem, from a freight as well as a passenger standpoint. Previously, the plans considered had for their only object the establishment of a convenient terminus in New York, to avoid the delays and difficulties involved in the necessity of transporting passengers and freight across the North River. When the Long Island Railroad became practically a part of the Pennsylvania System, it was possible and desirable to extend the project so as to provide, not only for a great prospective local traffic from all parts of Long Island, but also for through passenger and freight traffic to the New England States, and to and from all points on the Pennsylvania System, thus avoiding the long ferriage from Jersey City around the harbor to the Harlem River.

This paper has for its subject the New York Tunnel Extension project, and is intended merely as an introduction to the detailed accounts of the construction of the various divisions of the line to be given in succeeding papers prepared by the engineers who actively carried out the work. The project, however, forms the most important part of the comprehensive scheme adopted by the Pennsylvania Railroad Company for conducting its traffic into and through New York City, and a brief description of this general plan is therefore necessary in order that the relations of the tunnel line to the other parts of the transportation project may be clearly understood.

General Plan for Traffic Facilities at New York.

The component elements of the general plan outlined by the late A. J. Cassatt, President, in his open letter to the Board of Rapid Transit Railroad Commissioners of the City of New York, dated January 18th, 1906, are indicated on Fig. 1, and may be briefly summarized as follows:

1.—The Pennsylvania Tunnel and Terminal Railroad, generally referred to as the New York Tunnel Extension of the Pennsylvania Railroad. This line begins near Newark, N. J., crosses the Hackensack Meadows, and passes through Bergen Hill and under the North River, the Borough of Manhattan, and the East River to the large terminal yard, known as Sunnyside Yard, in Long Island City, Borough of Queens, New York. The line will be more fully described elsewhere.

2.—The electrification of the Long Island Railroad within the city limits.

3.—The Pennsylvania freight terminal yard and piers at Greenville, N. J., connecting by ferry with the Bay Ridge terminal of the Long Island Railroad.

4.—The Bay Ridge Improvement of the Long Island Railroad from East New York to Bay Ridge.

5.—Yards for increasing the freight facilities in the Boroughs of Brooklyn and Queens.

6.—The Atlantic Avenue Improvement in Brooklyn, involving the removal of the steam railroad surface tracks and the extensive improvement of the passenger and freight station at Flatbush Avenue.

7.—The New York Connecting Railroad, extending through a part of the Borough of Queens and crossing the East River by a bridge at Ward's and Randall's Islands to Port Morris, N. Y.

8.—The Glendale Cut-Off of the Long Island Railroad.

9.—New piers and docks in Newtown Creek at its confluence with the East River.

10.—Electrification of the United Railroads of New Jersey Division from Newark to Jersey City.

The parts sustained by these elements in the work of transportation and distribution are briefly as follows:

The New York Tunnel Extension is essentially a passenger line, although the Company has not only the legal powers but also the

facilities for making it a through route for freight if desired. It will transport passengers to and from the centrally located station at 33d Street and Seventh Avenue in New York City, joining the Long Island System at Sunnyside Yard, and, by means of the New York Connecting Railroad, it will form a link in the through traffic line, connecting the whole Pennsylvania System with the New England States. This line has been designed for the safe and expeditious handling of a large volume of traffic. The requirements include handling the heaviest through express trains south and west from the main line as well as the frequent and lighter local-service trains. For through service the locomotive principle of operation has been adhered to, that is, electric locomotives will take up the work of the steam locomotives at the interchange yard at Harrison, N. J., and, for excursion and suburban service to nearby towns, provision will be made for electric locomotives, or by operation of special self-propelled motor cars in trains, the project being planned to give the greatest flexibility in method of operation to meet the growing demand in the best way.

The New York Connecting Railroad has important functions both for freight and passenger service. When constructed it will be about 12 miles long, and will form a part of the line to the New England States for through passenger and freight service, and also carry local freight to and from Sunnyside Yard and Brooklyn, and all points on Long Island. By means of this line it will be possible to make the Brooklyn station at Flatbush Avenue a station on the through System for New England as well as the Western States.

The initial equipment of the Western Division of the Long Island Railroad for electric traction has been made in advance of the opening of the tunnel line in order to take care of the requirements of the Atlantic Avenue improvement. This improvement involved the elimination of grade crossings within the City of Brooklyn and the conversion of the railroad line which was previously on the surface of the streets to part subway and part elevated line from the Flatbush Avenue Terminal to East New York Station, a distance of 5¼ miles. One of the requirements of this improvement was that the motive power should be changed to some form of power not involving combustion. This led to the adoption of electricity, and, in order to meet operating necessities, involved the electrification of connecting lines beyond the improvement proper, so that local service could be handled to the end of the

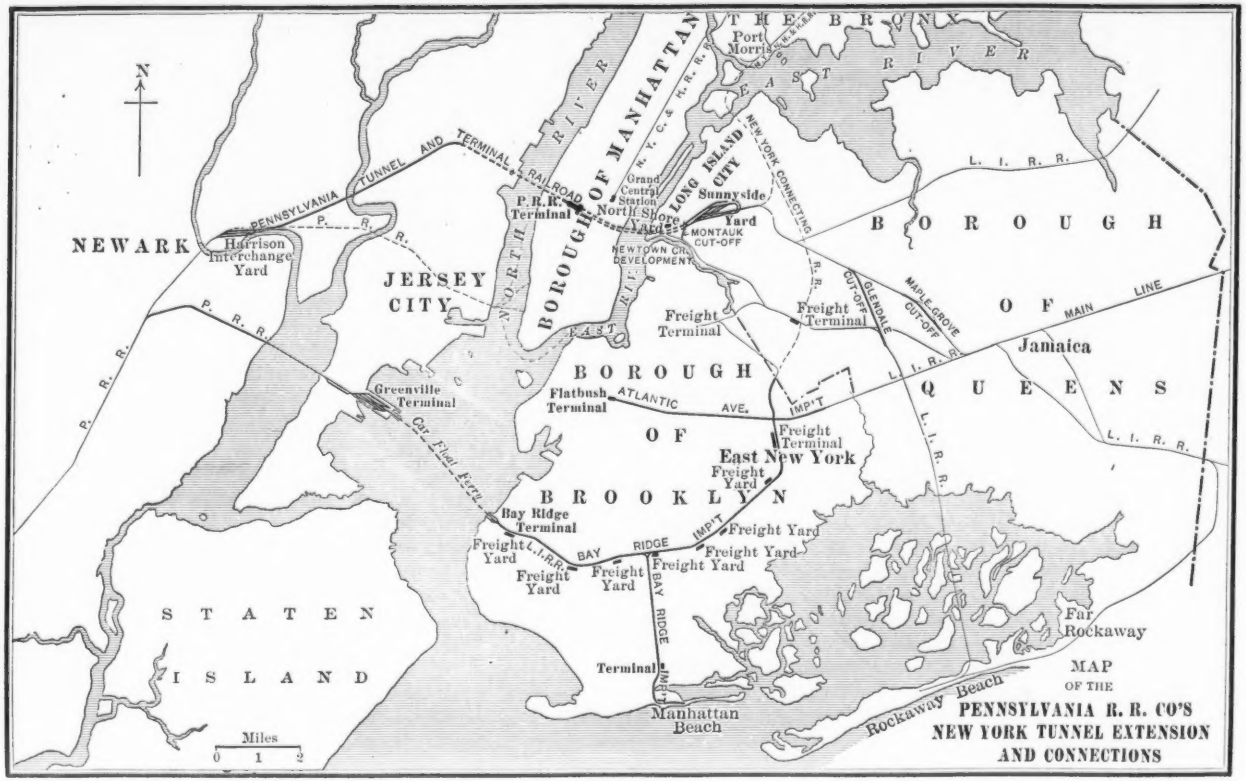

N
NEWARK
Harrison Interchange Yard
PENNSYLVANIA TUNNEL AND TERMINAL RAILROAD
P. R. R.
JERSEY CITY
Greenville Terminal
Cor. Fleet Ferry
STATEN ISLAND
Miles
0 1 2
NORTH RIVER
BOROUGH OF MANHATTAN
P.R.R. Terminal
Grand Central Station
North Shore Yard
N. Y. C. & H. R. R. R.
NEWTOWN CR. DEVELOPMENT
LONG ISLAND CITY
Sunnyside Yard
NEW YORK CONNECTING
MONTAUK CUT-OFF
Port Morris
N. Y. W. & B. R. R.
THE BRONX
EAST RIVER
L. I. R. R.
BOROUGH OF QUEENS
MAIN LINE
Jamaica
GLENDALE CUT-OFF
MAPLE GROVE CUT-OFF
Freight Terminal
Freight Terminal
EAST
BOROUGH OF BROOKLYN
Flatbush Terminal
ATLANTIC AVE.
IMP'T
Freight Terminal
East New York Freight Yard
Bay Ridge Terminal
Freight Yard
L. I. R. R.
BAY RIDGE
IMP'T
Freight Yard
Freight Yard
Freight Yard
Freight Yard
Freight Yard
BAY RIDGE
Terminal
IMP'T
Manhattan Beach
Rockaway Beach
Far Rockaway
MAP OF THE PENNSYLVANIA R. R. CO'S NEW YORK TUNNEL EXTENSION AND CONNECTIONS

FIG. 1.

runs without changing the motive power. The extent of the electrification thus required was found to be about 100 single-track miles. This extensive electrification work was undertaken and completed in the summer of 1905, upon the completion of the Atlantic Avenue improvement proper, and since that time has been in successful operation. On the near approach of the construction of the New York Terminal improvement, plans for additional electrification on the Long Island Railroad were made, and the work is now in progress on the extensive additions required to couple up the tunnel extension with the various lines centering at the Long Island City terminus.

The Bay Ridge Improvement of the Long Island Railroad comprises the readjustment of the right of way and the establishment of new grades in order to do away with grade crossings from the freight terminal at Bay Ridge to a junction with the New York Connecting Railroad at East New York, a distance of 10.4 miles. It also provides for the re-location of the line and the elimination of grade crossings on the branch running to Manhattan Beach, a distance of 3.7 miles. The work is being executed without interrupting traffic, and in all about 75 grade crossings will be abolished. This improvement became necessary in order to provide for the rapid extension of population into the suburban districts and for the present and future requirements of the section, to establish municipal conveniences and facilities, and to open additional streets across the right of way. To accomplish these ends, the line has been built in cuts and on embankments, there being about 6.4 miles of the former, 3.3 miles of the latter, and a tunnel, 3 500 ft. long, where the line crosses the Atlantic Avenue improvement.

The Atlantic Avenue improvement, as mentioned above, involved the removal of the railroad tracks from the street surface for a distance of about 5¼ miles. This was done by constructing a series of elevated and subway structures, there being about 2.1 miles of the former, 2.4 miles of the latter, and 0.8 mile of approaches, eliminating more than 90 grade crossings. In the light of recent developments, it may be of interest to note that one of the reasons for establishing a combination elevated and subway line was that, at the time the improvement was projected, no underground railroad in the country, of similar length and carrying a heavy volume of local traffic, was operated by electricity, and public sentiment was against the operation of the entire length of

the line underground by steam power. This improvement also provided for depressing the entire Flatbush Avenue station and a freight yard. As the work progressed, the original plans for the station were greatly enlarged, the remodeled station covering about 61 city lots.

The main point of passenger distribution is the New York station. Other important stations will be Flatbush Avenue, Brooklyn; Jamaica, Long Island, where the changes to and from electric motive power will be made; and Newark, N. J. Many other places, including the seaside resorts on Long Island and in New Jersey, will feel the benefits of the direct tunnel railroad into and through New York City. The Glendale Cut-Off will materially shorten the route and running time from New York through the tunnels to Rockaway Beach.

The plans contemplate that passengers to and from the lower part of Manhattan will be carried by the steam line between Newark and Jersey City and cross the North River by ferry or the Cortlandt Street tunnels of the Hudson Company. Eventually, the old main line will be electrified and supersede the steam service between Newark and Jersey City.

The Greenville Yard is the most important point for the receipt, transmission, and distribution of freight. From this point freight can be transported, without breaking bulk, by a comparatively short car-ferry to the Long Island Railroad terminus at Bay Ridge, and thus a very large part of the Pennsylvania Railroad Company's floatage in New York Harbor and the East River will be abolished, the floatage distance being reduced in the case of the New England freight from about 12 to 3 miles. This traffic will be routed from Bay Ridge *via* the Long Island Railroad to a connection with and thence over the New York Connecting Railroad to the New York, New Haven and Hartford Railroad at Port Morris, N. Y.

As the facilities for the handling of freight in the Boroughs of Brooklyn and Queens had become insufficient for taking care of the prospective traffic, eleven new local delivery yards, having a combined area of about 2 153 city lots, have been established, and three existing yards are to be improved and enlarged so as to give a combined area of about 687 city lots. Of these new yards, the Bay Ridge freight terminal, containing about 790 city lots, is the largest; its functions have been described above. There is a freight terminal at East New York 200 ft. wide and a mile long, containing about 566 city lots,

which will be the distributing point of freight for the entire East New York section. This yard is depressed, and will be crossed by six viaducts carrying city streets. The North Shore freight yard, containing 109 city lots, is connected with the Montauk Division by an overhead construction, known as the Montauk Freight Cut-Off, whereby all freight traffic to Jamaica may be kept out of the way of the Jamaica passenger traffic from the tunnels.

It may be of interest to indicate briefly how much has already been accomplished in the execution of this general plan, and what still remains to be done for its completion.

The larger part of the electrification of the Long Island Railroad and the elimination of grade crossings within the built-up city limits, the Atlantic Avenue improvement, and the yard and piers at Greenville, have been completed. The Sunnyside Yard and the Glendale Cut-Off will be completed during the next twelve months. On the Tunnel and Terminal Railroad the embankment and bridge work across the Hackensack Meadows and all the tunnels and excavation from the west side of Bergen Hill to Long Island City, except a short section near the eastern end of the line, have been completed. The New York station and other buildings and facilities connected therewith are well advanced. The laying of the track, the electrification of the line, and the installation of the signaling and lighting systems are under way. It is anticipated that the line will be ready for operation in the spring of 1910.

Report has been made to the Public Service Commission that a large part of the right of way for the New York Connecting Railroad has been obtained, and more than $3 000 000 has been spent by this railroad. The piers and docks at Newtown Creek and the electrification of the line from Newark to Jersey City are not yet actively under way.

Estimated Cost of the Improvements.

As appears from the foregoing statement, only parts of the improvements contemplated in the general scheme have been completed, others are in progress, and others have not yet been commenced. It is therefore impossible at the present time to make a close estimate of the total expenditure involved in the execution of the entire scheme. The following estimate of the cost of the Pennsylvania Railroad Company's

improvements in the New York District when fully completed is based on the best information now available:

New York Tunnel Extension and Station, including Interchange Yards at Harrison, N. J., and Sunnyside, L. I., P. T. & T. R. R. Co.	$100 000 000
Long Island Railroad electrification, Bay Ridge and Atlantic Avenue improvements, Glendale Cut-Off, freight yards, and new equipment	35 000 000
New York Connecting Railroad, to be built jointly by the Pennsylvania R. R. Co. and the New York, New Haven and Hartford R. R. Co., about	14 000 000
Pennsylvania Railroad improvements in the State of New Jersey, electrification of line from Jersey City to Park Place, Newark, Greenville freight line and terminal on New York Bay	10 000 000
Total	$159 000 000

Corporate Organization and Franchise Conditions.

As the tunnel extension lies partly in the State of New Jersey and partly in the State of New York, it was necessary to charter two companies, each covering the territory within the State to which it belonged. The New Jersey corporation was entitled the Pennsylvania, New Jersey and New York Railroad Company, and the New York corporation, the Pennsylvania, New York and Long Island Railroad Company. These organizations were completed early in 1902. Subsequently, after the tunnels had been joined under the North River, the companies were consolidated, on June 26th, 1907, and thereby formed the present company under the name of the Pennsylvania Tunnel and Terminal Railroad Company, a corporation of both States.

Mr. Cassatt, President of the Pennsylvania, New York and Long Island Railroad Company, made application in its behalf for a franchise to extend the lines of the Pennsylvania Railroad by tunnels under the North River to a passenger station to be erected in New

York City and thence under the East River to a connection with the Long Island Railroad, on May 5th, 1902.

The franchise for that part of the tunnel line which is within the State of New York, that is, from the boundary line between New York and New Jersey, in the Hudson River, to the eastern terminus at Sunnyside Yard, Long Island, is contained in the certificate issued by the Board of Rapid Transit Railroad Commissioners of the City of New York on October 9th, 1902.

The essential features of the franchise have been summarized substantially as follows in the report of the Committee of the Board of Rapid Transit Railroad Commissioners of the City of New York, dated June 14th, 1902:

First.—A grant by the city in perpetuity of rights, subject, however, to a periodic re-adjustment of payments at intervals of twenty-five years, as follows:

(*a*) To construct and operate a railroad of two tracks from the boundary between New York and New Jersey under the Hudson River opposite the westerly foot of Thirty-first Street, Borough of Manhattan, thence running under the Hudson River and Thirty-first Street to the East River and under the East River to a terminus in Queens Borough. The Company is permitted on notice within ten years to give up the right to these two tracks.

(*b*) A like right for a railroad of two tracks beginning near the same point under the Hudson River, thence running under Thirty-second Street to the East River, and under that river to the terminus in Queens Borough, with a right for two additional tracks in Thirty-second Street, west of Ninth Avenue, and one additional track between Seventh and Fifth Avenues in Manhattan.

(*c*) A like right for a railroad of two tracks beginning at the station terminal site at Thirty-third Street and Seventh Avenue and thence running under Thirty-third Street and the East River to the terminal in Queens Borough, with a right for one additional track on Thirty-third Street, between Seventh and Fifth Avenues.

(*d*) A right to maintain a terminal station occupying the four blocks bounded by Thirty-first Street, Seventh Avenue, Thirty-third Street and Ninth Avenue, the lots on the east side of Seventh Avenue between Thirty-first and Thirty-third Streets, and the underground portions of Thirty-first and Thirty-third Streets, between Seventh and Eighth Avenues and between

Eighth and Ninth Avenues, the Company having itself acquired the land included in such four blocks and lots on the east side of Seventh Avenue.

(*e*) To occupy for such terminal facilities all of Thirty-second Street lying between the westerly side of Seventh Avenue and the easterly side of Eighth Avenue, and between the westerly side of Eighth Avenue and the easterly side of Ninth Avenue. As soon as the statutory right of the city authorities to make the conveyance shall be put beyond doubt the Railroad Company is obliged to buy such two portions of Thirty-second Street, which will then become completely dedicated to the purposes of their station. (These portions of Thirty-second Street were subsequently purchased by the Railroad Company.)

(*f*) To have along such routes the necessary facilities for the operation of passenger and freight trains, including telegraph wires and the various wires and cables for the distribution of power, heat, and light.

Second.—The requirement of the consent of the Mayor, the Board of Aldermen, the Board of Estimate and Apportionment, and the other authorities of the city having control of the streets.

Third.—The obligation of the Pennsylvania Company to begin construction within three months after obtaining the necessary consents and complete the railroad within five years after construction shall begin, except the route under Thirty-first Street, for the completion of which the company is allowed ten years after the completion of the remainder of the railroad.

Fourth.—Payments by the Pennsylvania Company for the first twenty-five years, as follows: A rental of $200 per annum for the right to occupy land under the Hudson and East Rivers outside of pier lines. A rental for ground within pier lines and for underground portions of streets in Manhattan Borough, at fifty cents per linear foot of single track per annum, for the first ten years, and during the next fifteen years one dollar per annum per linear foot. A rental for ground within pier lines and for underground portions of streets in Queens Borough at one-half the rates payable for Manhattan Borough. A rental for underground portions of Thirty-first and Thirty-third Streets, between Seventh and Eighth Avenues, and between Eighth and Ninth Avenues (such portions extending almost up to the surface, except under the south sidewalk of Thirty-first Street and north sidewalk of Thirty-third Street) at $14 000 per annum for the first ten years, and at $28 000 per annum for the next fifteen years.

For the portions of Thirty-second Street, between Seventh and Eighth Avenues, and between Eighth and Ninth Avenues, when the statutory power of the city to make a sale shall be put beyond doubt,

the city is to sell and the Railroad Company is required to buy such portions for the sum of $788 600. The rentals for river and track rights begin at the date of operation. For the underground spaces under Thirty-first and Thirty-third Streets, used for station extension, the rentals begin at the commencement of construction, or when the company entered thereon.

Such annual payments may be summarized as follows:

	First 10 years.	Next 15 years.
For river rights	$200.00	$200.00
For tunnel rights in Manhattan Borough, being 44 341 ft. (partly estimated) of single track	22 170.00	44 341.00
For tunnel rights in Queens Borough, being 8 100 ft. (partly estimated) of single track	2 025.00	4 050.00
For street rights on Thirty-first and Thirty-third Streets, north and south of terminal	14 000.00	28 000.00
In all, per annum	$38 395.00	$76 591.00

If the route under Thirty-first Street be availed of, these amounts will be increased by $16 652.50 for the first ten years, and by $33 305 for the next fifteen years.

The amounts to be paid are to be readjusted at the end of twenty-five years; and thereafter at intervals of twenty-five years. If the city and the Railroad Company shall not agree upon the readjusted rates, they are to be determined by the Supreme Court of this State.

Fifth.—The railroad to be entirely in tunnel except where it approaches the surface at its eastern terminal near Thomson Avenue, in Queens Borough. The uppermost part of the tunnel is to be at least nineteen feet below the surface of the street; but this limitation does not apply to the portions of Thirty-first and Thirty-third Streets opposite the terminal station between Seventh and Ninth Avenues, where the Company may occupy the underground portions of the street under the roadway to within thirty inches of the surface, and under the sidewalks on Thirty-first and Thirty-third Streets opposite to the station to within five feet of the surface, the company to properly care for sewers, water, gas and other pipes and underground structures lawfully in the street.

Sixth.—The company to make good all damage done to property of the city by its construction work or operations, and to abutting owners all damage done through any fault or negligence of the company, or of any contractor or sub-contractor engaged upon its work of construction or operation. The Tunnel Company to keep Thirty-first and Thirty-third Streets opposite the station well paved with smoth pavement and in thoroughly good condition.

Seventh.—Tunnel excavations to be done without disturbing the surface of the street, except in the portions of Thirty-first and Thirty-third Streets, and Seventh, Eighth and Ninth Avenues in front of the terminal station, and except in Queens Borough, with the power to the Rapid Transit Board, wherever conditions elsewhere make surface excavation necessary for efficient construction, to grant the right for such excavation, subject to conditions to be then prescribed by the Board. The tracks are to be constructed of the most approved plan so as to avoid noise or tremor. All plans for, and the method of doing, the work are made subject to the approval of the Rapid Transit Board.

Eighth.—The motive power to be electricity, or such other power not involving combustion as may be approved by the Board.

Ninth.—The company to have no power to carry on merely local traffic, except with the approval of the Board and for additional consideration to be paid the city. Traffic is defined as local which begins and ends in the city within five miles of the terminal station on Seventh and Ninth Avenues.

Tenth.—The railroad to be diligently and skillfully operated, with due regard to the convenience of the traveling public.

Eleventh.—The city to have a lien upon the franchise and real property of the company to secure the payment of rental.

Twelfth.—The rights of the city to be enforceable by action for specific performance, or mandamus, or otherwise.

Thirteenth.—The company not to oppose the construction of any rapid transit railroad along or across the same routes which do not actually interfere with the authorized structures of the company.

Fourteenth.—The city to have an ample right of inspection of the railroad, and to enter upon it for examination, supervision, or care of city property, or for other purposes.

Fifteenth.—The company to be bound to maintain and strengthen all parts of its railways under streets or avenues so that the same shall support safely any structures superimposed or which may hereafter be superimposed thereon by the city or under public authority.

Sixteenth.—The company to have the right to convey or mortgage the franchise, but every grantee, whether directly or under a mortgage, to assume the obligations already assumed by the Railroad Company and the Railroad Company not to be relieved of such obligations by the grant.

This franchise was passed by the Board of Aldermen on December 16th and approved by the Mayor on December 23d, 1902.

Subsequently, an agreement, dated June 21st, 1907, was entered into by the City of New York, the Tunnel Company, and the Long Island Railroad Company covering the construction of the Sunnyside Yard, which forms the eastern terminus of the line.

In pursuance of this agreement, the map or plan of the City of New York was changed by discontinuing or closing portions of fifty streets or avenues, and by changing the grades of sixteen streets or avenues, in the Borough of Queens, and the portions of streets and avenues thus discontinued and closed, most of which were not opened for public use, were sold to the Railroad Companies. The agreement, however, reserved to the City permanent and perpetual underground rights and easements to maintain in a reasonable manner, not inconsistent with the construction and operation of the railroad facilities of the Companies, its existing sewers, drains, and other sub-surface structures in, under, and through the lands within the lines of the discontinued portions of each of such streets and avenues, including the right to repair, rebuild, and enlarge the same, and to construct in a reasonable manner, not inconsistent with the construction and operation of the railroad facilities of the Companies, such additional sewers or drains in, under, or through the lands as may be hereafter required by the City, together with the right to enter upon the premises from time to time as may be necessary for the purpose of inspecting, repairing, constructing, or rebuilding the sub-surface structures.

The agreement required the Companies to construct at their expense, four viaducts or bridges over their tracks and terminal development, three with roadways 42 ft. wide, one with a roadway 60 ft. wide, and each to have two sidewalks 10 ft. wide, the work to include the paving of the roadways and sidewalks.

The Companies are further required to pay one-half the cost of the construction of the foundations, abutments, piers, superstructures, and approach of an additional viaduct or bridge over the Sunnyside Yard, to have a roadway not more than 60 ft. wide and two sidewalks each 10 ft. wide, and to grant the City of New York a perpetual easement for the continuance of the same in the location upon which it shall be constructed.

The agreement further provides that the Companies shall not injure the sewers or other substructures now existing or hereafter constructed under the streets and avenues, and, in case of injury, that they shall repair them or pay the cost thereof; that the viaducts shall be completed within the shortest time consistent with their safe and proper construction, and that during their construction temporary streets shall be provided for the accommodation of traffic.

The Companies are required to bear all the expense of changes of grade in the streets and avenues, except those made necessary by the construction of the viaduct or bridge to be paid for in part by the City; to indemnify the City against all liability for any and all damages which may accrue on account of any street which may be closed or the grades of which may be changed in pursuance of the agreement; to assume all liabilities by reason of the construction or operation of the railroads, or the construction of the viaducts, and to save the city harmless from any liability whatever, to either persons or property, by reason of the construction or operation of the railroads or the construction of the viaducts.

The Companies are also required to indemnify the City against and pay the cost of all alterations which may be required to the sewerage or drainage system or to any sub-surface structures and pipes laid in the streets or avenues on account of the construction and operation of the terminal, passenger yard, or freight yard of the Companies, or on account of the changes in grades or street system.

The Companies are authorized, if they deem it necessary to the construction or to the efficient operation of the terminal passenger yard or freight yard, to depress, at their expense, any pipes or other sub-surface structures now under the surface of any of the portions of the streets or avenues discontinued or closed, or to elevate and carry the same upon any of the viaducts or bridges, the plans of such depression or elevation to be approved by the Board of Estimate and Apportionment.

All works within, upon, or over the public streets and avenues are subject to the supervision and inspection of the proper municipal officer or officers, under such regulations as he or they may determine and be authorized by law to impose; and the plans for the construction of viaducts or bridges are to be approved by the Board of Estimate and Apportionment.

The Companies are required to cede to the City of New York perpetual easements for the right to continue and maintain the viaducts or bridges over the streets and avenues, sufficient for their control by the City for the purpose of police regulation and other control contemplated by the City ordinances for the case of streets or highways; reserving, however, the right to construct and maintain, at their own expense, such connections between the viaducts or bridges and their

property as shall not interfere with the use of the viaducts or bridges for street purposes.

The Companies are also required to cede to the City, grade and curb, portions of five existing or proposed streets or avenues, and to pave portions of two other avenues.

Mr. A. J. Cassatt, President of the Pennsylvania Railroad Company, was President of the Companies constituting the New York Tunnel Extension until his death on December 28th, 1906, and Mr. James McCrea, President of the Pennsylvania Railroad Company, was elected his successor, and is now President of the Pennsylvania Tunnel and Terminal Railroad Company.

Mr. Samuel Rea, Second Vice-President of the Pennsylvania Railroad Company, has served as Vice-President since the incorporation of the enterprise.

Mr. A. J. County has been Assistant to the President since June 26th, 1907, and prior thereto and from the incorporation of the tunnel enterprise served as Secretary of the Pennsylvania, New Jersey and New York Railroad Company and as Assistant Secretary of the Pennsylvania, New York and Long Island Railroad Company, which, as heretofore stated, constitute the Pennsylvania Tunnel and Terminal Railroad Company.

Engineering Organization.

Mr. Rea, Vice-President, has general charge of all matters involved in the designing and execution of the project.

The Board of Engineers.—Before the beginning of the work, the Management appointed a Board of Engineers which was instructed to examine into the New York Tunnel Extension project; to pass upon the practicability of the undertaking; to determine upon the best plans for carrying it out; to make a careful estimate of its cost; and, if the work was undertaken, to exercise general supervision over its construction.

President Cassatt's letter appointing the Board contains the following further instructions:

"You are requested to procure all additional information that may be needed, sparing neither time nor any necessary expense in doing so, for I am sure it is not necessary for me to say that, in view of the magnitude and great cost of the proposed construction, and of the

novel engineering questions involved, your studies should be thorough and exhaustive, and should be based upon absolute knowledge of the conditions."

The Board was organized on January 11th, 1902, when it held its first session, and continued in the performance of its duties until April 30th, 1909, when it was dissolved, its work having been completed.

The Board held regular and special sessions to receive progress reports from the Chief Engineers in direct charge of construction, and to consider questions relating to the plans and details of the work submitted by its members or referred to by the Management. It then reported its conclusions to the Vice-President for approval before the work was undertaken.

The Management earnestly impressed upon the Board throughout the whole period of its labors, that the Tunnel Extension and facilities were to be designed and constructed without regarding cost as a governing factor, the main considerations being safety, durability, and proper accommodation of the traffic. No expenditure tending to insure these conditions was to be avoided.

The Board, when organized, was composed as follows: Col. Charles W. Raymond, Corps of Engineers, U. S. Army, Chairman, Messrs. Gustav Lindenthal, Charles M. Jacobs, Alfred Noble, and William H. Brown.

Mr. George Gibbs was appointed a member of the Board on April 9th, 1902. Mr. Lindenthal resigned on December 15th, 1903, and Mr. Brown resigned on March 1st, 1906. Mr. Rea and all the members of the Board are members of the American Society of Civil Engineers, and Mr. Noble is a Past-President of the Society.

Mr. William R. Mead, of the firm of McKim, Mead, and White, Architects for the Terminal Station, was associated with the Board for the consideration of architectural subjects.

Mr. Robert H. Groff, Secretary of the Company, was also Secretary of the Board until his resignation on January 31st, 1907. Mr. William Couper was Acting-Secretary from April 15th, 1907, to April 30th, 1909.

S. Johannesson, Assoc. M. Am. Soc. C. E., was Engineer Assistant to the Chairman from December 1st, 1905, to April 30th, 1909.

Division of the Work.—For the purposes of actual construction, the line was divided into four parts: the Meadows Division, the North

River Division, the Terminal Station, and the East River Division. A chief engineer appointed by the Management had charge of the construction of each Division. The chief engineers exercised full authority in the organization of the working forces, and in the general conduct and management of the work of construction on their respective Divisions, in accordance with the plans for such work approved by the Board of Engineers and the Management.

Architects were employed to design the Terminal Station building and superintend its erection; and structural engineers to design and erect steel structures and facilities, and carry on the work under the direction of a Chief Engineer of the Company.

Committees, consisting principally of officers of the Pennsylvania Railroad Company, co-operating with the regular engineering organization, were appointed to consider the operating features of the project, so that the experience of the Pennsylvania Railroad Company's organization might be utilized in the work.

Description of the Line.

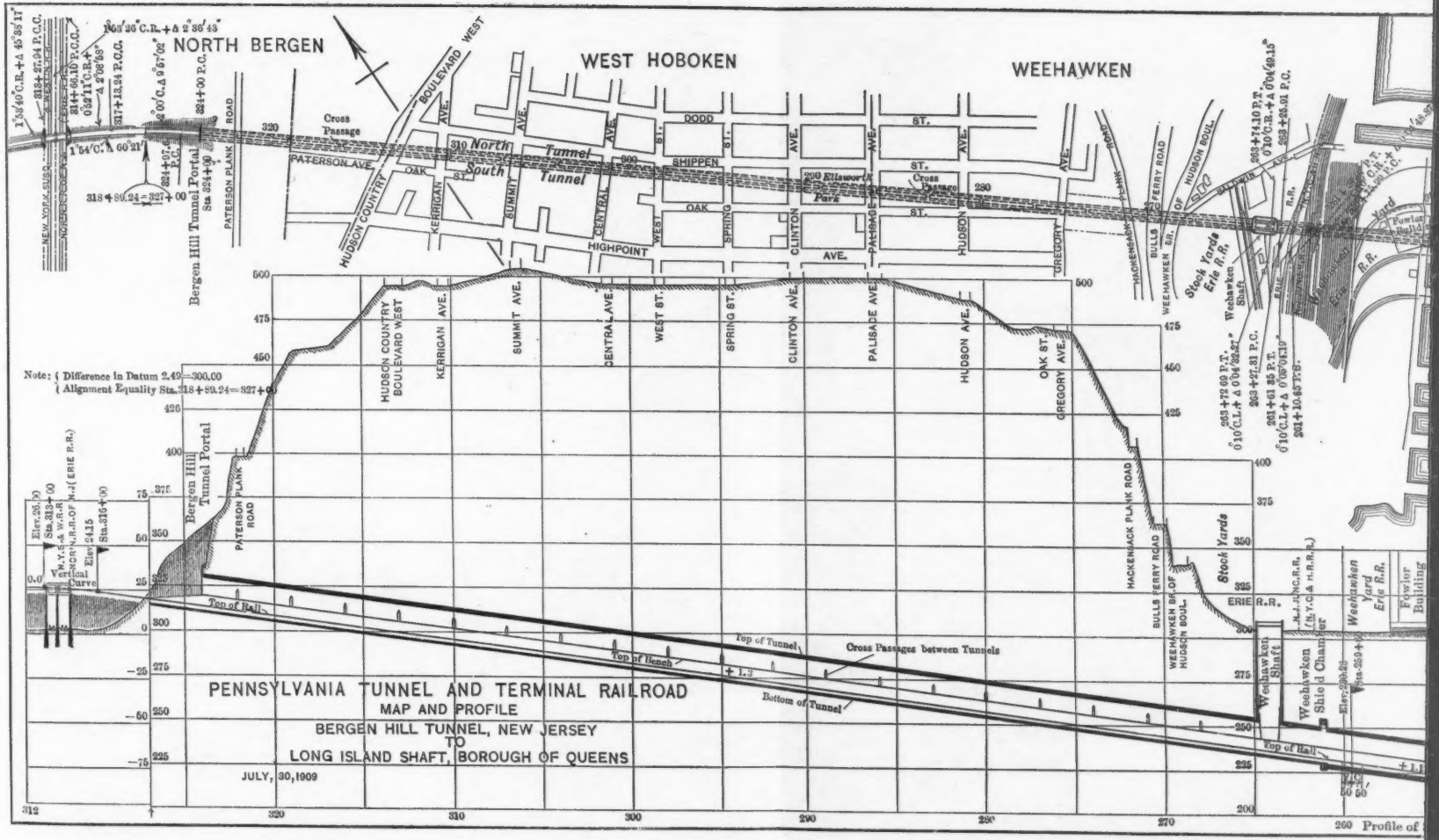
The following summary description of the various divisions of the line is intended to give a comprehensive idea of the general features of the project. Full details will be given in succeeding papers. The line and its respective divisions are shown on Plate LVIII.

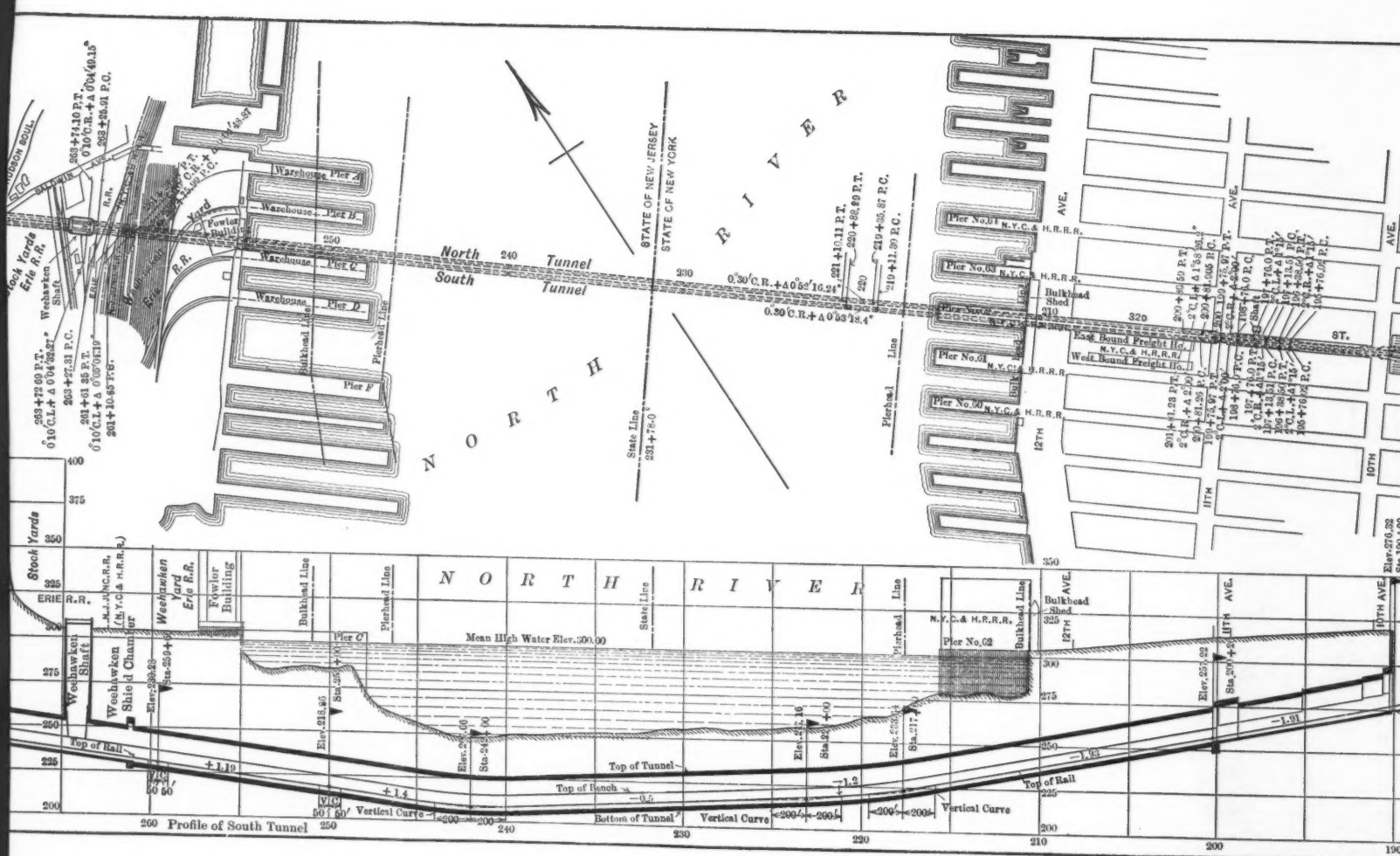
Meadows Division.—Chief Engineer until March 1st, 1906, Mr. William H. Brown, Chief Engineer, Pennsylvania Railroad Company, when he retired from active service with the latter Company; since March 1st, 1906, Mr. Alexander C. Shand, Chief Engineer, Pennsylvania Railroad Company.

This Division consists of an "interchange yard" at Harrison, near Newark, N. J., adjoining the tracks of the present New York Division of the Pennsylvania Railroad, and a double-track railroad across the Hackensack Meadows to the west side of Bergen Hill, a distance of 6.04 miles. The construction is embankment and bridge work, including bridges across the Pennsylvania, Erie, and Lackawanna Railroads, and the Hackensack River.

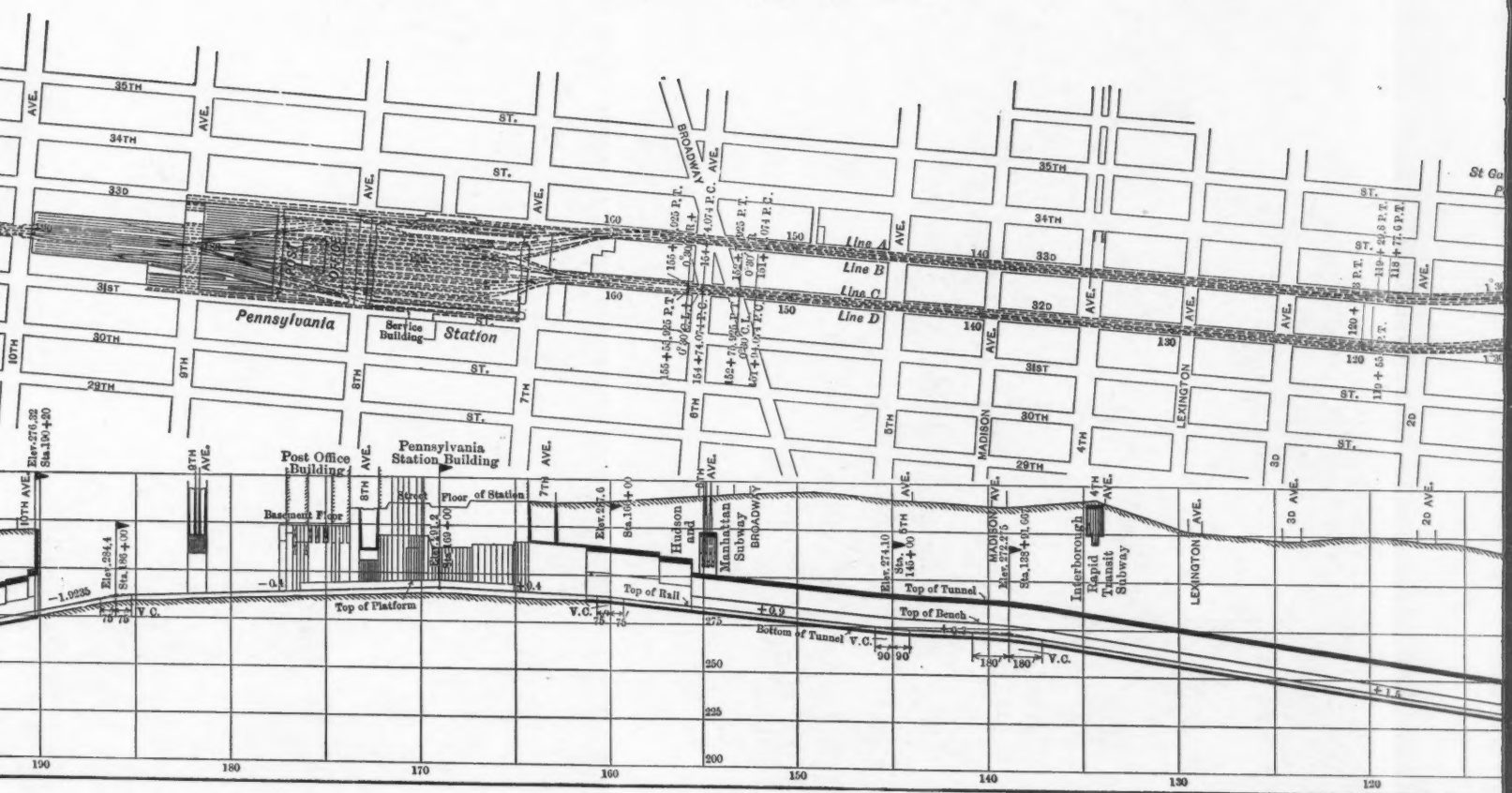
North River Division.—Chief Engineer, Mr. Charles M. Jacobs.

This Division commences at the west side of Bergen Hill and passes through the hill in two single-track rock tunnels to a large permanent shaft at Weehawken, near the west shore of the North River,


NORTH BERGEN
WEST HOBOKEN
WEEHAWKEN
Bergen Hill Tunnel Portal Sta 324+00
PATERSON PLANK ROAD
Cross Passage
PATERSON AVE.
BOULEVARD WEST
HUDSON COUNTRY
KERRIGAN AVE.
SUMMIT AVE.
CENTRAL AVE.
WEST ST.
SPRING ST.
CLINTON AVE.
PALISADE AVE.
HUDSON AVE.
GREGORY AVE
OAK ST.
DODD ST.
SHIPPEN ST.
HIGHPOINT
OAK ST.
North Tunnel
South Tunnel
Ellsworth Park
Cross Passage
HACKENSACK PLANK ROAD
BULLS FERRY ROAD
WEEHAWKEN BR. OF HUDSON BOUL.
Stock Yards Erie R.R.
Weehawken Shaft
Weehawken Yard Erie R.R.
Fowler Building
Note: (Difference in Datum 2.49=300.00
(Alignment Equality Sta. 318+80.24=327+00
Bergen Hill Tunnel Portal
Elev. 25.00
Sta. 313+00
W.Y.S. & W.R.R.
Elev. 24.15
Sta. 316+00
N.J. JUNC. R.R. (N.Y.C. & H.R.R.R.)
Top of Rail
Top of Tunnel
Top of Bench
+ 1.3
Bottom of Tunnel
Cross Passages between Tunnels
PENNSYLVANIA TUNNEL AND TERMINAL RAILROAD
MAP AND PROFILE
BERGEN HILL TUNNEL, NEW JERSEY
TO
LONG ISLAND SHAFT, BOROUGH OF QUEENS
JULY, 30, 1909
Profile of


NORTH RIVER
STATE OF NEW JERSEY
STATE OF NEW YORK
North Tunnel
South Tunnel
Warehouse Pier A
Warehouse Pier B
Warehouse Pier C
Warehouse Pier D
Pier F
Fowler Building
Bulkhead Line
Pierhead Line
State Line
Pier No.51 N.Y.C. & H.R.R.R.
Pier No.53 N.Y.C. & H.R.R.R.
Pier No.61 N.Y.C. & H.R.R.R.
Pier No.60 N.Y.C. & H.R.R.R.
Bulkhead Shed
East Bound Freight Hg.
West Bound Freight Hg.
Weehawken Shaft
Weehawken Shield Chamber
Weehawken Yard Erie R.R.
Stock Yards
Erie R.R.
N.J.J.N.C.R.R.
(N.Y.C. & H.R.R.R.)
Mean High Water Elev. 0.00
Pier C
Pier No.62
Top of Rail
Top of Tunnel
Top of Bench
Bottom of Tunnel
Vertical Curve
Profile of South Tunnel
12TH
11TH
10TH
AVE.
ST.

BOROUGH OF MANHATTAN



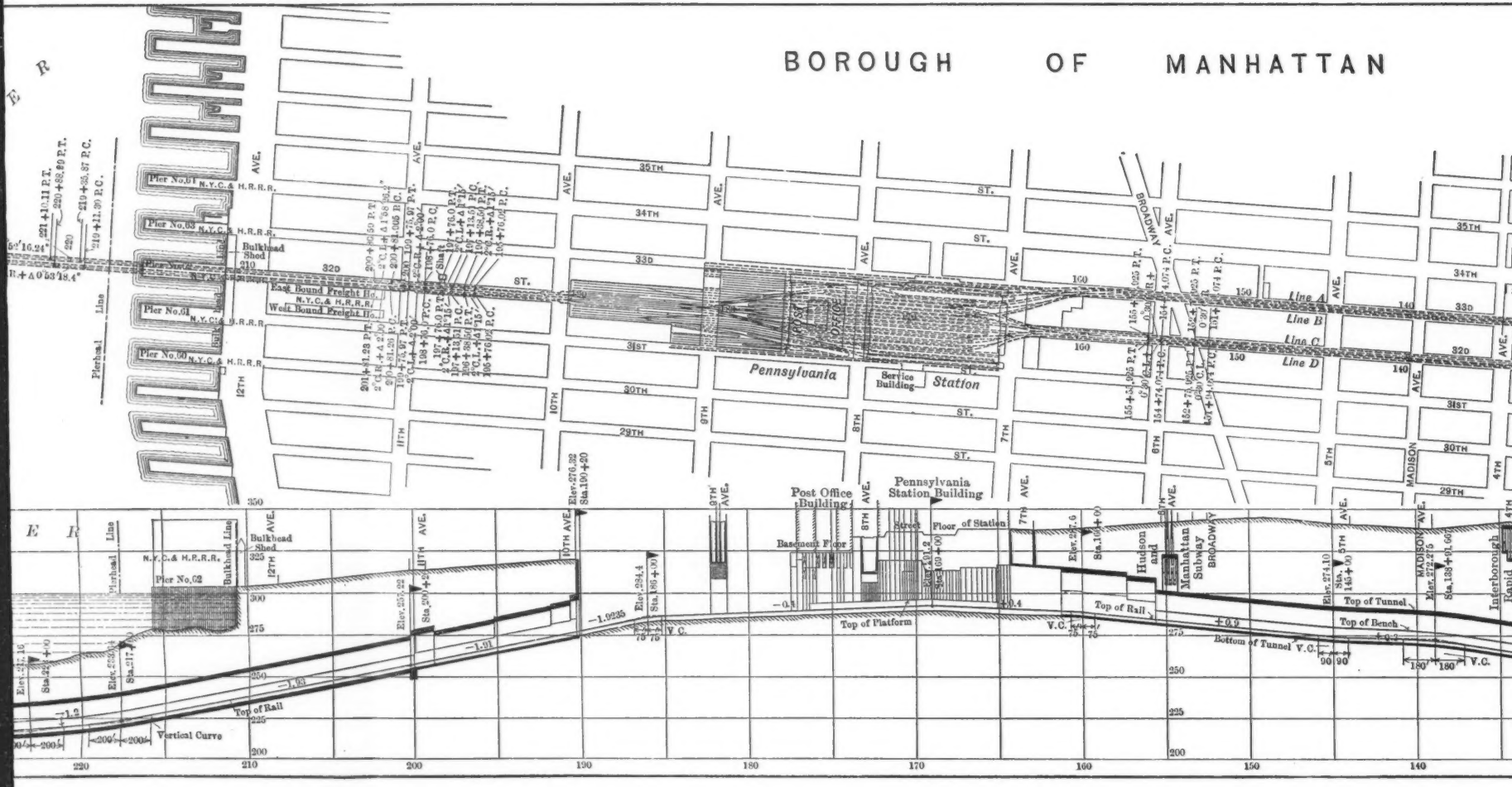
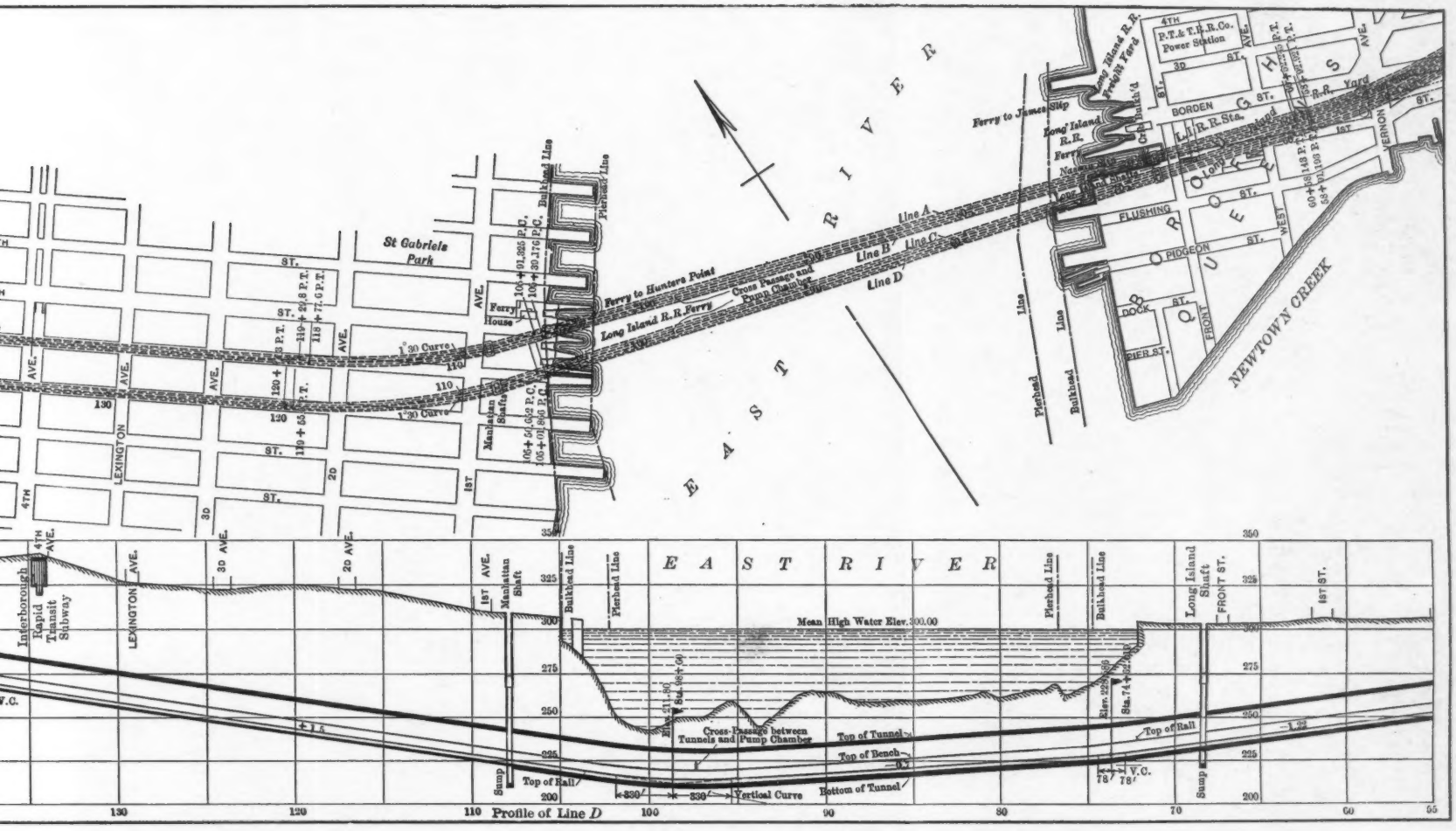

BOROUGH OF MANHATTAN
Pier No.01 N.Y.C.& H.R.R.R.
Pier No.63 N.Y.C.& H.R.R.R.
Pier No.61 N.Y.C.& H.R.R.R.
Pier No.60 N.Y.C.& H.R.R.R.
Bulkhead Shed
Pierhead Line
Bulkhead Line
East Bound Freight Ho.
N.Y.C.& H.R.R.R.
West Bound Freight Ho.
Pennsylvania Station
Service Building
Post Office Building
Pennsylvania Station Building
Basement Floor
Street Floor of Station
Hudson and Manhattan Subway
BROADWAY
MADISON AVE.
Line A
Line B
Line C
Line D
Pier No.62
Top of Platform
Top of Rail
Top of Tunnel
Top of Bench
Bottom of Tunnel
Vertical Curve
V.C.
Interborough Rapid

PLATE LVIII.
PAPERS, AM. SOC. C. E.
SEPTEMBER, 1909.
RAYMOND ON
NEW YORK TUNNELS: PENNSYLVANIA RAILROAD.



and thence eastward a distance of 224 ft. to the Weehawken shield-chamber. It then passes under the river through two cast-iron, concrete-lined, single-track tunnels, with outside diameters of 23 ft., to a point under 32d Street, near Eleventh Avenue, in New York City, and thence through two single-track tunnels of varying cross-section, partly constructed in cut-and-cover, to the east side of Tenth Avenue. It then passes into the Station Yard and terminates at the east building line of Ninth Avenue. The work included the Station Yard excavation and walls from Tenth Avenue to Ninth Avenue, and the retaining walls and temporary underpinning of Ninth Avenue. The aggregate length of the line in this Division is 2.76 miles.

New York Station and Approaches.—Mr. George Gibbs, Chief Engineer of Electric Traction and Station Construction.

The Station and its approaches extend from the east line of Tenth Avenue eastward to points in 32d Street and 33d Street, respectively, 292 ft. and 502 ft. east of the west line of Seventh Avenue. This Division included the construction of subways and bridges for the support of 31st and 33d Streets and Seventh, Eighth, and Ninth Avenues, the Station building between Seventh and Eighth Avenues, the foundations for the post office to be erected west of Eighth Avenue, the service power-house in 31st Street between Seventh and Eighth Avenues, the power-house in Long Island City, the traction system, tracks, signals, and miscellaneous facilities required in the physical construction of the entire terminal railroad ready for operation. Messrs. McKim, Mead, and White were the architects for the Station and Messrs. Westinghouse, Church, Kerr and Company executed the structural engineering work, both in the station and for the support of the streets, as well as the construction of the subways.

The station is of steel skeleton construction with masonry curtain walls, all supported by a system of columns extending to a rock foundation. This building covers two city blocks and one intersecting street, and has an area of about 8 acres. It is 774 ft. long, 433 ft. wide, with an average height above the street of 69 ft., and a maximum of 153 ft. The main waiting-room is 277 ft. long, 103 ft. wide and 150 ft. high. The Concourse is 340 ft. long and 210 ft. wide.

The level of the track system below the street surface varies from 39 to 58 ft., and is from 7 to 10 ft. below mean high water in the harbor, thereby necessitating the establishment of an elaborate system

of drainage over the entire station yard area. Access to the street is gained by elevators and stairways.

To accelerate the loading and unloading of the trains, high platforms will be constructed in the station on a level with the floors of the cars, in order to avoid the use of car steps and increase the traffic capacity of the station.

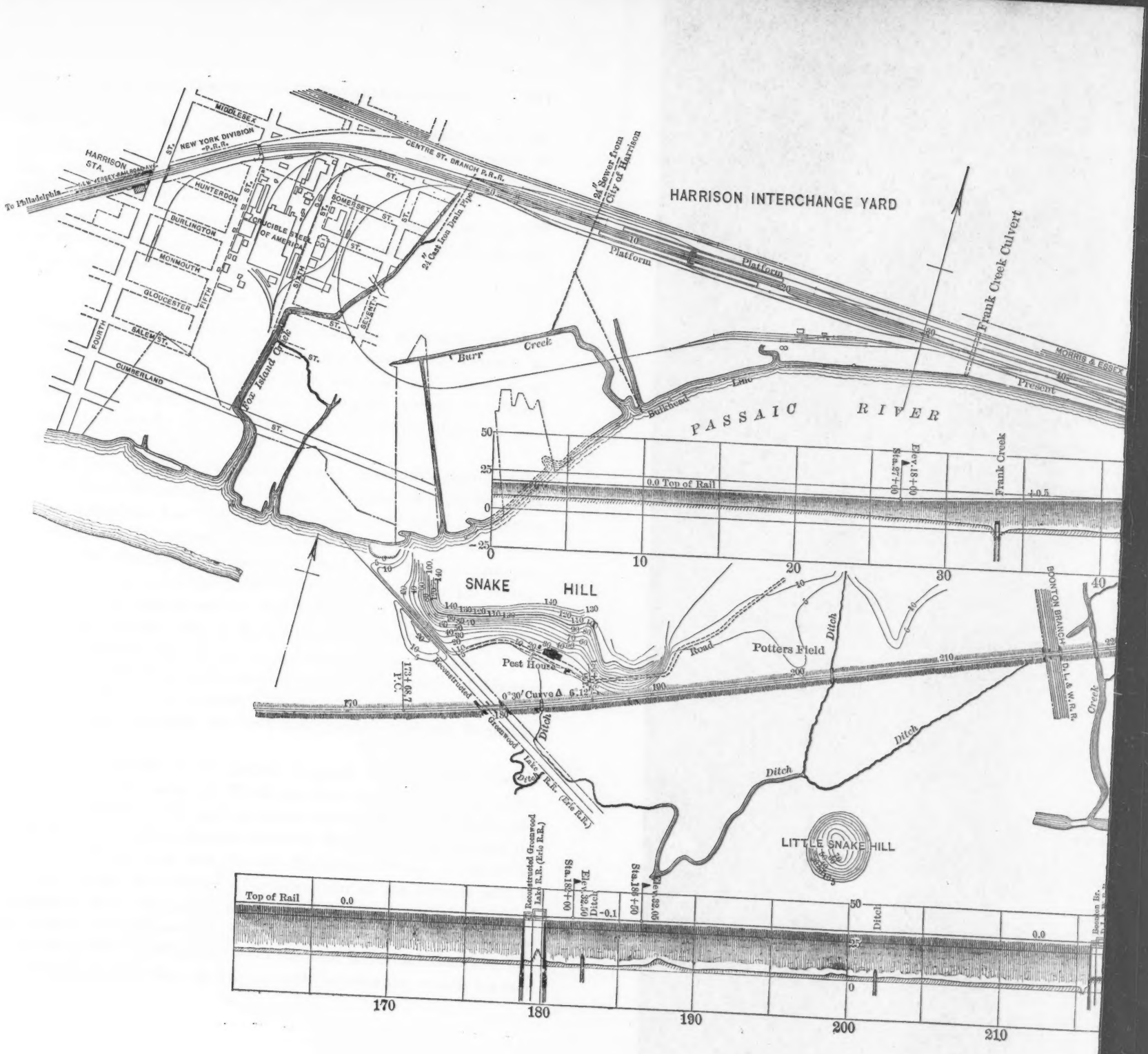
There will be 21 standing-tracks at the station, and 11 passenger platforms, providing 21 500 ft. of platform adjacent to passenger trains. Within the station area, which from Tenth Avenue to the normal tunnel sections east of Seventh Avenue comprises 28 acres, there will be a total of about 16 miles of track.

The service plant for the installation of machinery for lighting, heating, and ventilating the station, and for operating the interlocking system, is located in an independent building south of the station.

The Power-House to supply the electrical energy for the operation of the tunnel line and the Long Island Railroad is situated on property in Queens Borough adjoining the present Long Island Railroad Station near the East River, and was constructed under the Chief Engineer of Electric Traction and Station Construction. As at present designed, the dimensions of the structure are 200 ft. by 262 ft., outside measurement. It can accommodate six generating units of 5 500 kw., the standard adopted for future work, and two of 2 500 kw. for lighting the tunnels. The ultimate capacity of this station when extended will be about 105 000 kw.

East River Division.—Chief Engineer, Mr. Alfred Noble.

This Division begins at the eastern limits of the New York Station at a point in 32d Street, 292 ft. east of the west line of Seventh Avenue, and at a point in 33d Street, 502 ft. east of the west line of Seventh Avenue, and also includes the excavation work and retaining walls for the station site and yard, to the track level, westward to Ninth Avenue. It extends eastward from the station under 32d and 33d Streets through tunnels partly three-track and partly so-called twin tunnels to Second Avenue; thence the line curves to the left under private property to permanent shafts a few feet east of First Avenue. Four single-track, cast-iron, concrete-lined tunnels, with outside diameters of 23 ft., pass under the East River, and, after passing through permanent shafts near the bulkhead line, reach the surface in Long Island City from 3 000 to 4 200 ft. east of the East River. The tunnel portals are in


HARRISON INTERCHANGE YARD
PASSAIC RIVER
SNAKE HILL
LITTLE SNAKE HILL
Potters Field
Pest House
Road
Ditch
Burr Creek
Frank Creek Culvert
Frank Creek
Bulkhead Line
Present
Reconstructed Greenwood Lake R.R. (Erie R.R.)
Greenwood Lake R.R. (Erie R.R.)
Top of Rail
0.0 Top of Rail
Elev. 18+00
Elev. 27+00
Elev. 38.09
Elev. 32.30
Sta. 18+00
Sta. 18+20
-0.1
+0.5
0°30' Curve Δ 5°12'
P.C. 123+08.7
MIDDLESEX
NEW YORK DIVISION P.R.R.
CENTRE ST. BRANCH P.R.R.
HARRISON STA.
To Philadelphia
HUNTERDON
BURLINGTON
MONMOUTH
GLOUCESTER
SALEM ST.
CUMBERLAND
SOMERSET ST.
CRUCIBLE STEEL OF AMERICA
24" Sewer from City of Harrison
24" Cast Iron Drain Pipe
Fox Island Creek
MORRIS & ESSEX
Platform
BOONTON BRANCH
D. L. & W. R. R.
Creek
170
180
190
200
210

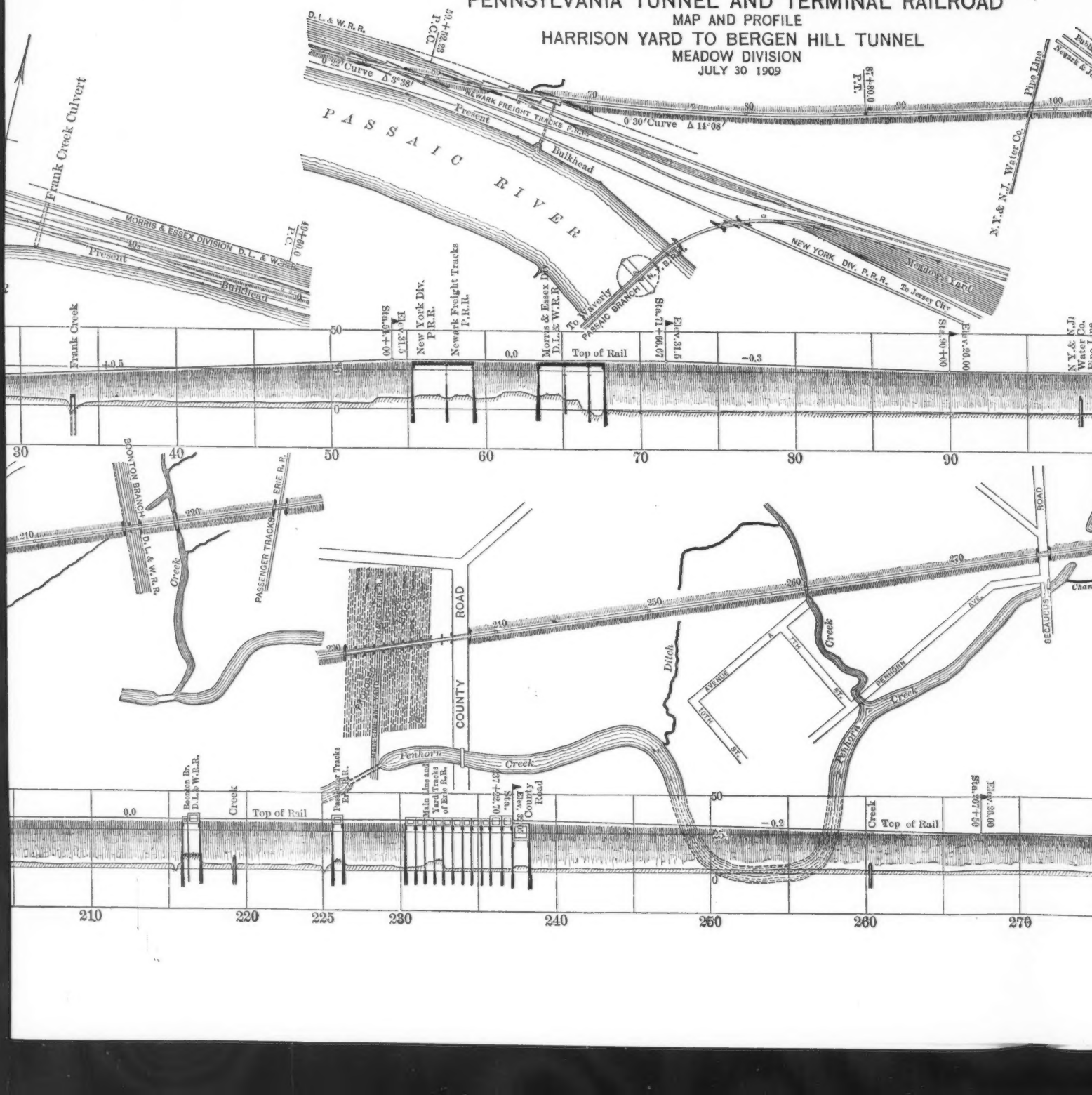
PENNSYLVANIA TUNNEL AND TERMINAL RAILROAD

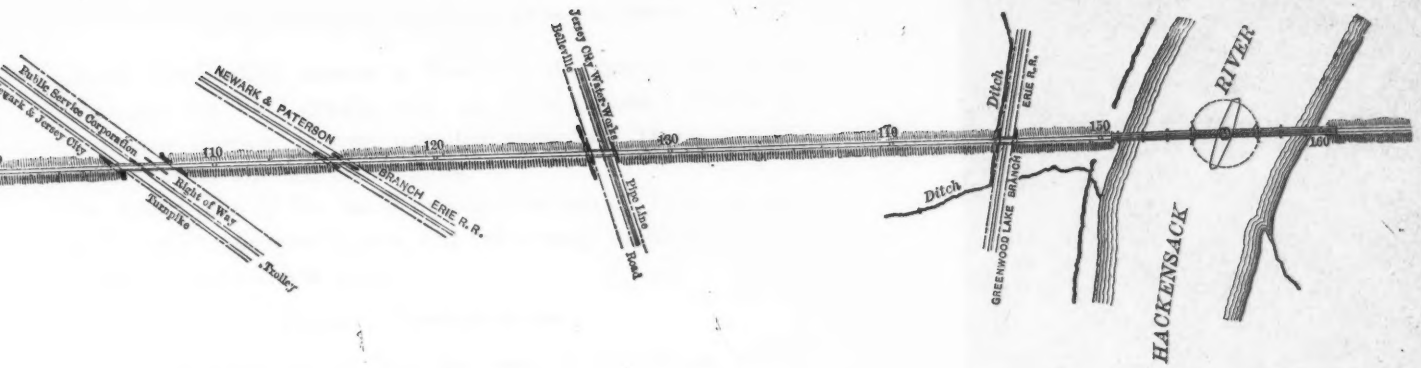
MAP AND PROFILE

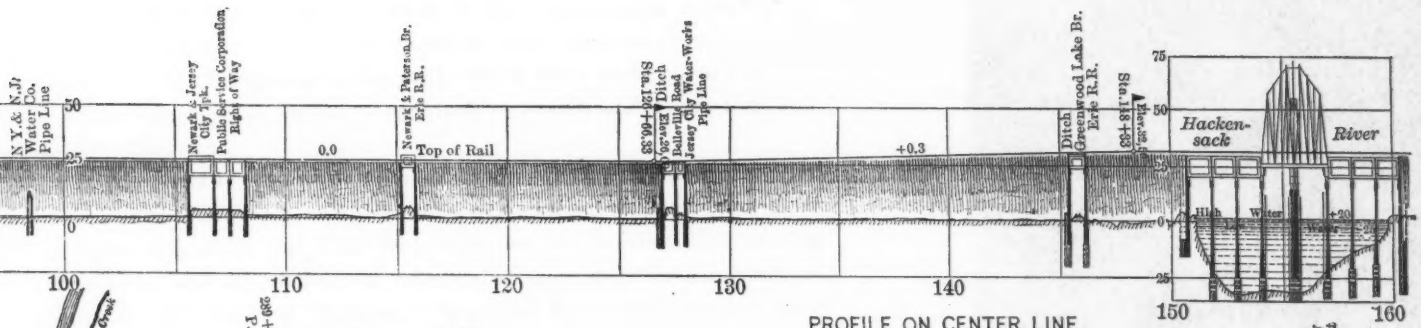
HARRISON YARD TO BERGEN HILL TUNNEL

MEADOW DIVISION

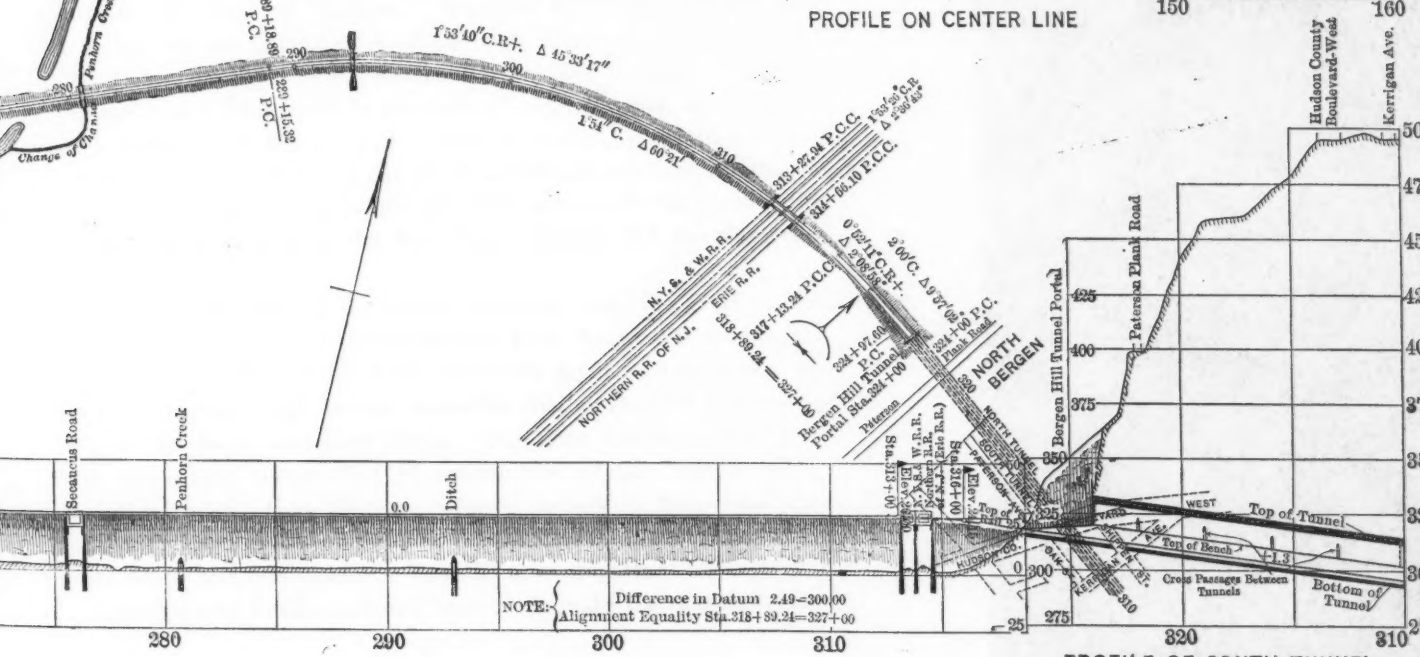
JULY 30 1909







PROFILE ON CENTER LINE



NOTE: Difference in Datum 2.49=300.00
Alignment Equality Sta. 318+89.24=327+00

PROFILE OF SOUTH TUNNEL

Sunnyside Yard, which extends to Woodside, the easterly end of the Division, and the Yard grading with its buildings and a number of City viaducts crossing it were executed under this Division. The total length of the Division is 4.48 miles.

The total length of the entire line is 13.66 miles. There are 6.78 miles of single-track tube tunnels, and the average length of the tunnels between portals is 5.56 miles.

General Considerations.

Details have been omitted from the foregoing description, as they can be treated better and more fully by the constructing engineers in succeeding papers. There are, however, some general considerations involved in the designing of the work, which may, perhaps, be referred to more conveniently in this introductory paper, and these will now receive attention.

In all parts of the work problems were encountered requiring for their solution large expenditures and much engineering skill; but many of these difficulties had been frequently met in previous engineering experience, and the methods of overcoming them were well understood. Thus, in the Meadows Division, a long and heavy embankment, part of which was on submerged meadow land, and many bridge foundations had to be constructed; in the Bergen Hill tunnels, very tough trap rock was encountered; in the tunnels under the city, the work was much complicated and its cost increased greatly by the necessity of caring for sewers, water and gas pipes, and the foundations of adjacent buildings; and many troublesome problems were met in the construction of the tunnels connecting the East River tunnels with the Sunnyside Yard.

The novel features of the project, however, were the great tunnels extending the line under the North and East Rivers. Tunnels of the kind contemplated, to be used for heavy and rapid railroad traffic, had never been constructed through materials similar to those forming the beds of the North and East Rivers. Questions arising in connection with the design and method of construction of the tunnels will be considered later. Here they are referred to only in their relation to the location and grades of the line, in which connection the conditions controlling their establishment were the most important elements.

Location and Grades.—It was desirable to make the tunnels between

the bulkhead lines of the rivers as straight as possible, and it was necessary to place them at sufficient depth below the dredging plane of the War Department (which in the North and East Rivers is 40 and 26 ft. below mean low water, respectively) to insure them against possible injury from heavy anchors or sunken vessels. Furthermore, they had to pass under the piers and bulkheads of Manhattan at a depth sufficient to make it certain that they would not affect the stability of those structures. Another consideration influencing the establishment of the depth of the tunnels below the bottoms of the rivers became important as soon as the method of construction by shields with compressed air was adopted, namely, the necessity of providing sufficient cover to guard, as far as possible, against blow-outs during construction.

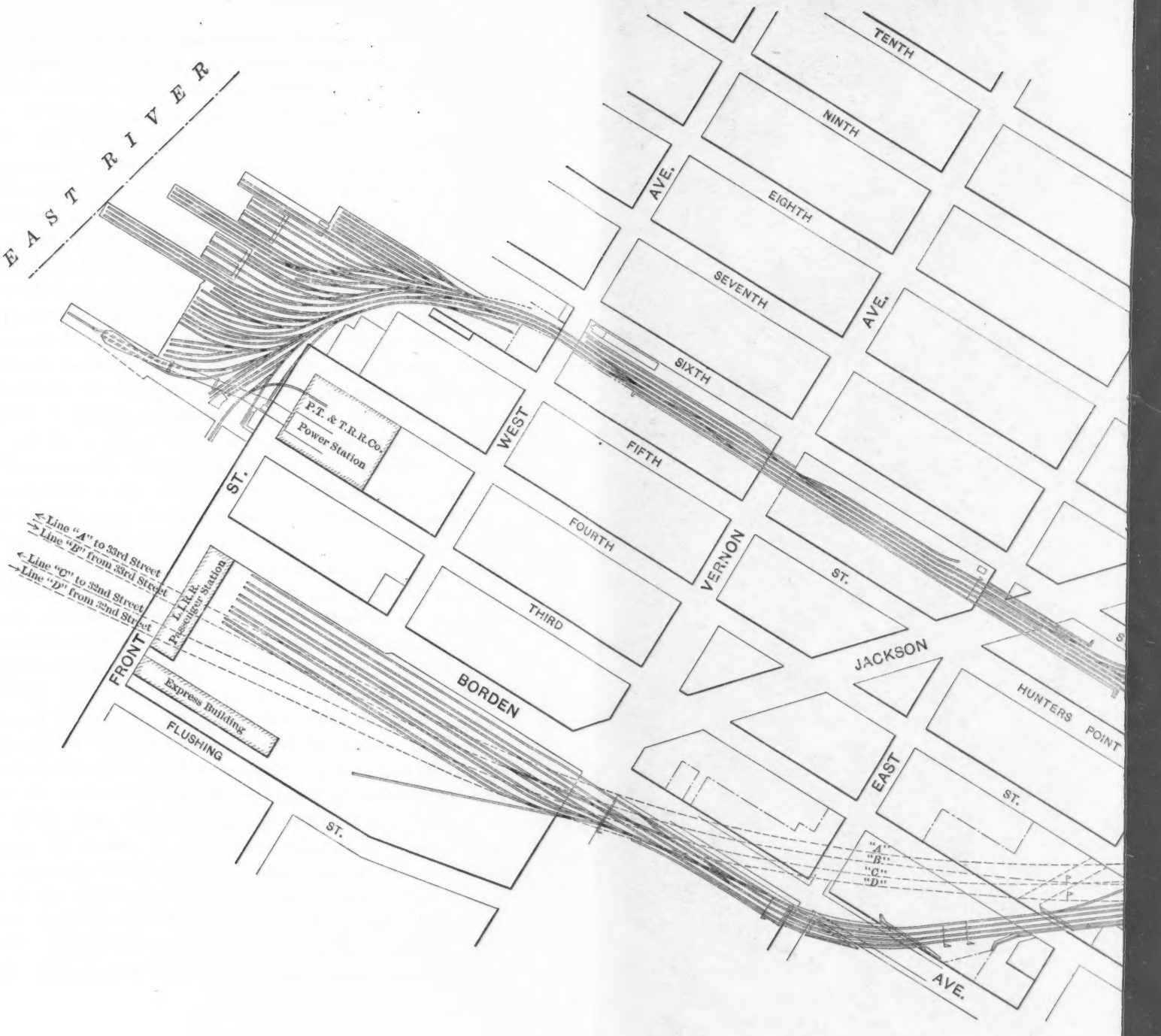
The tunnels under the city, connecting the sub-river tunnels with the Terminal Station were located so as to give as favorable grades as possible. The provision of the franchise requiring the tops of the tunnels to be at least 19 ft. below the street surface, which had been suggested by the Company to permit of future subways, had no effect on their location, as other conditions required them to be at a greater depth.

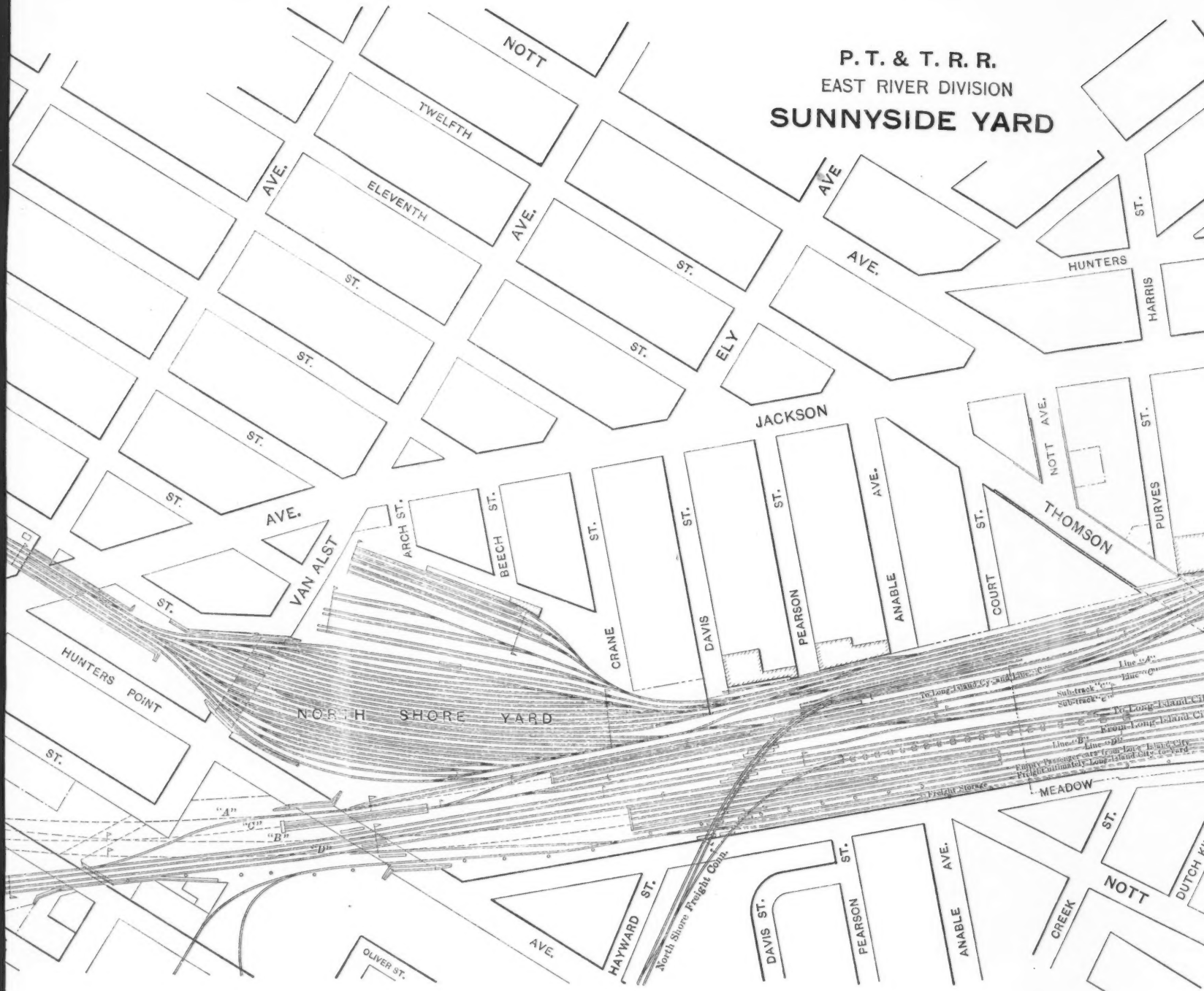
The line extending westward from Bergen Hill had to be established so as to give ample head-room at the numerous bridges over the railroads and highways which it crosses.

Eastward from the East River tunnels, the grades were established so as to rise as uniformly as possible to the level of the Sunnyside Yard.

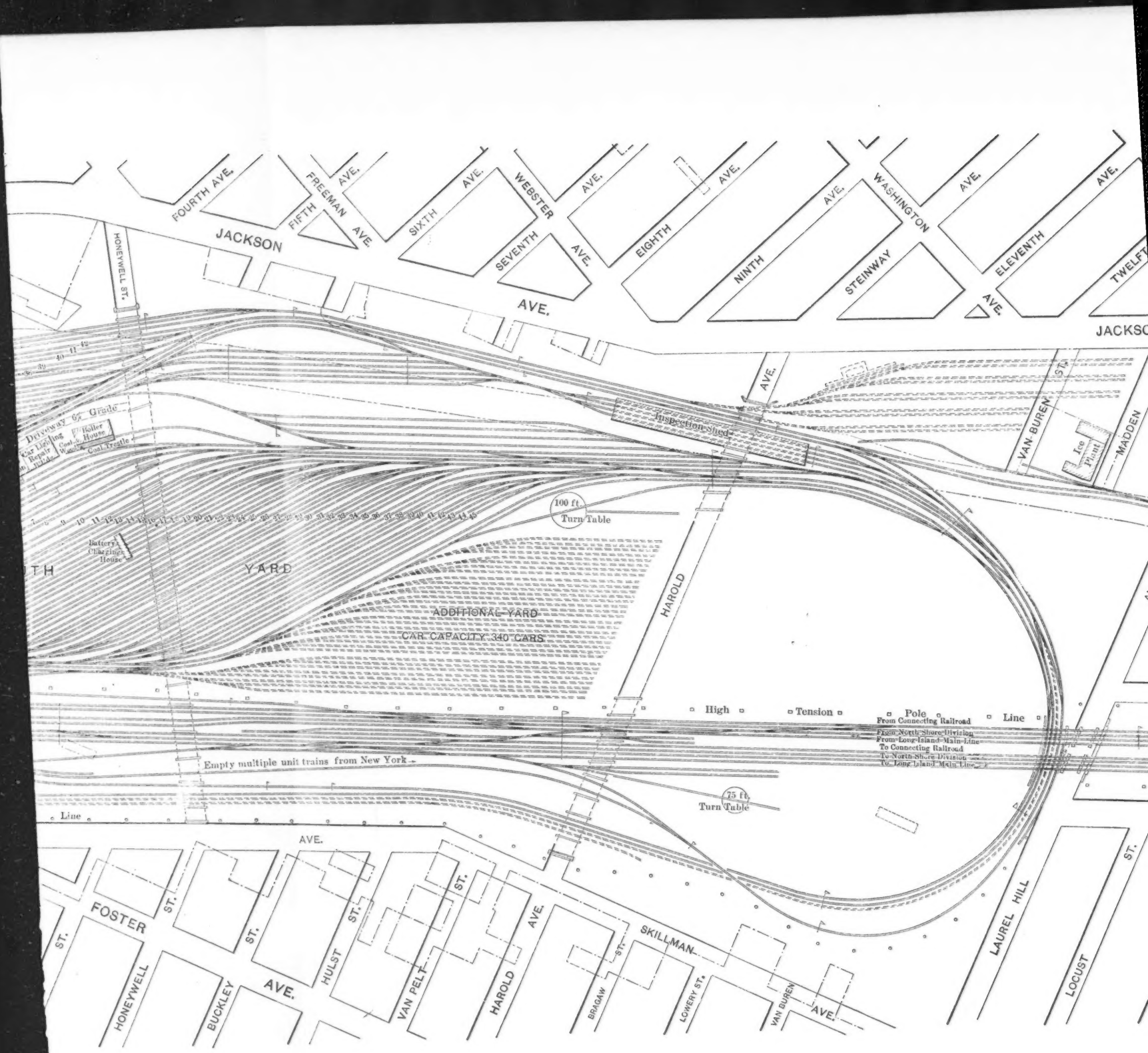
The general features of the line, as finally adopted and constructed, are as follows:

The maximum grade west of the Terminal Station occurs on the New York side of the North River, and is 2% in the west-bound and 1.93% in the east-bound tunnels. The ruling grades (for the ascending traffic) being 1.32% in the west-bound and 1.93% in the east-bound tunnels. In the tunnels east of the Terminal Station the ruling grade is 1.5% for both east-bound and west-bound traffic. There is, however, descending with the traffic, a short section on a grade of 1.9 per cent. These grades would be objectionable with steam locomotives under a heavy traffic, but the development of the electric locomotive has rendered possible the operation of grades which would have formerly been considered prohibitive.


EAST RIVER
TENTH
NINTH
AVE.
EIGHTH
SEVENTH
AVE.
SIXTH
WEST
ST.
FIFTH
FOURTH
VERNON
ST.
THIRD
JACKSON
BORDEN
HUNTERS POINT
FRONT
EAST
ST.
FLUSHING
ST.
AVE.
P.T. & T.R.R.Co. Power Station
L.I.R.R. Passenger Station
Express Building
← Line "A" to 33rd Street
→ Line "B" from 33rd Street
← Line "C" to 32nd Street
→ Line "D" from 32nd Street
"A"
"B"
"C"
"D"


P. T. & T. R. R.
EAST RIVER DIVISION
SUNNYSIDE YARD
NOTT
TWELFTH
ELEVENTH
AVE.
ST.
HUNTERS
HARRIS
ELY
JACKSON
NOTT AVE.
PURVES
THOMSON
VAN ALST
ARCH ST.
BEECH
CRANE
DAVIS
PEARSON
ANABLE
COURT
HUNTERS POINT
NORTH SHORE YARD
MEADOW
HAYWARD
OLIVER ST.
North Shore Freight Conn.
CREEK
DUTCH
"A"
"B"
"C"
"D"

QUEENSBORO BRIDGE APPROACH
ST.
ACADEMY
SO. WASHINGTON PL.
PAVONIA AVE.
SECOND AVE.
FOURTH AVE.
MIDDLEBURG PL.
HONEYWELL ST.
PROSPECT
RADDE
AVE.
HENRY ST.
SKILLMAN PL.
DUTCH KILLS ST.
QUEENS ST.
ORCHARD ST.
BARN ST.
BRIDGE
YARD
NORTH
SOUTH
Driveway 6% Grade
Sub Station
Car Lighting
Repair Shop
Boiler House
Coal & Water
Coal Trestle
Carpet Shed
Pullman Stores and Commissary
P.R.R. Commissary
P.R.R. Stores and Lavatories
Shelter
Umbrella Shed
Umbrella Shed
Battery Charging House
Platform
Platform
Platform
Sub "b"
Sub "d"
Sub "f"
Line "A" to 33rd Street
Line "C" to 32nd Street
To Long Island City
From Long Island City
Line "B" from 33rd Street
Line "D" from 33rd Street
Empty mul
Western Union Pole Line
SKILLMAN
ST.
DUTCH KILLS PL.
QUEENS PL.
ORTON ST.
MANLEY ST.
MOUNT ST.
SCHOOL ST.
VAN DAM
AVE.
APPROACH
HILL ST.
RAWSON ST.
MOORE ST.
FOSTER
HONEYWELL
BUCKLEY ST.


FOURTH AVE.
FIFTH AVE.
FREEMAN AVE.
SIXTH AVE.
WEBSTER AVE.
SEVENTH AVE.
EIGHTH AVE.
NINTH AVE.
WASHINGTON AVE.
STEINWAY AVE.
ELEVENTH AVE.
JACKSON AVE.
HONEYWELL ST.
Driveway 6% Grade
Car Lighting
Car Repair
Roller House
Coal & Sand House
Coal Trestle
Battery Charging House
Inspection Shed
VAN BUREN ST.
Ice Plant
MADDEN
100 ft. Turn Table
YARD
ADDITIONAL YARD
CAR CAPACITY 340 CARS
HAROLD
High Tension Pole Line
From Connecting Railroad
From North Shore Division
From Long Island Main Line
To Connecting Railroad
To North Shore Division
To Long Island Main Line
Empty multiple unit trains from New York
75 ft. Turn Table
AVE.
FOSTER ST.
HONEYWELL ST.
BUCKLEY AVE.
HULST ST.
VAN PELT ST.
HAROLD AVE.
BRAGAW ST.
SKILLMAN ST.
LOWERY ST.
VAN BUREN AVE.
LAUREL HILL
LOCUST ST.

PIERCE AVE.
TWELFTH AVE.
THIRTEENTH AVE.
FOURTEENTH AVE.
GRAHAM AVE.
FIFTEENTH AVE.
SIXTEENTH AVE.
SEVENTEENTH AVE.
EIGHTEENTH AVE.
NINETEENTH AVE.
TWENTIETH AVE.
NEWTOWN AVE.
TWENTY FIRST AVE.
ROAD
JACKSON AVE.
MADDEN ST.
AVE.
LOCUST ST.
Pond
GROVE ST.
Pumping Station No.3
BLISS ST.
CAROLIN ST.
AVE.
HEISER ST.
FITTING ST.
STONE ST.
DICKINSON ST.
AVE.
N. Y. & Q. Co. R. R. Car Barn
Western Union Pole Line
DREYER AVENUE
BARNETT
MIDDLEBURG ST.
GROVE ST.
BLISS ST.
CAROLIN AVE.
GOSMAN AVENUE
HEISER ST.
FITTING ST.
STONE ST.
DICKINSON ST.
AVENUE
Temporary Pole Line from "A" to "G"
Proposed Freight Yard
WOODSIDE

From the junction with the Pennsylvania Railroad, near Harrison, N. J., to Woodside, Long Island, a distance of 13.66 miles, there is an average of 1.5 curves per mile; the line having a total curvature of 230 degrees. The maximum curvature is 2 degrees.

Method of Construction of Sub-River Tunnels.—The character of the material through which the tunnels were to be constructed differed greatly in the two rivers. The bed of the North River, at the level of the tunnels, consists of silt composed principally of clay, sand, and water, while that of the East River is formed of a great variety of materials, such as quicksand, sand, boulders, gravel, clay, and bed-rock. When the method of construction had to be decided there were no thoroughly satisfactory precedents to follow in the case of either river, although the Gas Tunnel under the East River, the partly constructed Hudson Tunnels under the North River, the St. Clair Tunnel under the St. Clair River, the Blackwall and several other tunnels under the Thames River at London, supplied much useful information. The smaller tunnels for a lighter traffic, since so successfully constructed under the North and East Rivers, had not then been completed. Under these circumstances, it was the desire of the Management that the Board should receive and consider proposed methods of construction from all available sources; and during the first year of its labors much of its time was devoted to the examination and discussion of projects submitted for its consideration by engineers and practical builders, some of these projects having decided merit. Most of the methods proposed involved temporary structures, or the use of floating plant, in the navigable channels of the river. This was objectionable in view of the resulting obstruction to the enormous river traffic. After full consideration of the subject, it was decided to adopt the shield method with compressed air for the construction of the tunnels under both rivers, this being the only method recommended by the Chief Engineers, and having the great advantage of conducting all operations below the bottom of the river, thus avoiding obstruction of the channel.

Experience has shown, as was anticipated, that it is much more difficult to construct tunnels in such material as occurs in the East River and on the New Jersey side of the North River, than in more homogeneous material such as is found in the greater part of the North River. During the progress of construction under the East River, there were frequent blow-outs through fissures opened in the

river-bed, and the bottom of the river over the tunnel had to be blanketed continuously with clay, to check the flow of the escaping air.

In view of the serious difficulties which it was thought might be encountered in the application of the shield method to the East River work, other methods for the execution of this part of the project received special consideration, one of the methods considered being the freezing process. It was proposed to drive a small pilot tunnel and freeze the ground for a sufficient distance around it by circulating brine through a system of pipes established in the tunnel. The pilot tunnel was then to be removed and the full-sized tunnel was to be excavated in the frozen material and its lining placed in position. By this means, it was intended to avoid the danger incident to the use of compressed air in material of greatly varying character. This method contained too many elements of uncertainty to justify its adoption; but as the Management considered it desirable to have, if possible, an alternative method, an extended experiment was made with the freezing process. A pilot tunnel, 7 ft. 6 in. in diameter, was driven in the bed of the East River for a distance of 160 ft., circulating pipes were established in it, and brine at a very low temperature was passed through the pipes until the ground was frozen for a distance of about 11.5 ft. around the tunnel. Observations to determine the rate of cooling and other important points connected with the process were carefully made. When it was found that the construction of the tunnels was progressing satisfactorily by the shield method, and that so much time was required to freeze the material that the freezing process could not be used to advantage in this particular case, the experiment was discontinued.

Design of the Sub-River Tunnels.—The sub-river tunnels consist of a circular cast-iron shell, of the segmental, bolted type, having an outside diameter of 23 ft., lined with concrete having a normal thickness of 2 ft. from the outside of the shell. Through each plate of the shell there is a small hole, closed with a screw plug, through which grout may be forced into the surrounding material. Each tunnel contains a single track. A concrete bench, the upper surface of which is 1 ft. below the axis of the tunnel, is placed on each side of the track, the distance between benches being 11 ft. 8 in. These benches contain ducts for carrying electric cables. The main reason for adopting single-track tunnels instead of a larger tunnel containing two tracks was to

avoid the danger of accidents due to the obstruction of both tracks by derailment or otherwise. The tunnels are made just large enough to allow the passage of a train with perfect safety, as it was believed that with such an arrangement thorough ventilation would be secured by the motion of the trains. Experience seems to justify this assumption, but, in order to assure thorough ventilation under unusual conditions, such as the stoppage of trains in the tunnels, a complete ventilating plant will be provided for each tunnel. The rapidity and safety of construction were increased by making the tunnel as small as possible, one of the difficulties in the shield method of construction being the difference in hydrostatic pressure between the top and bottom of the shield, which increases with the diameter of the tunnel.

The concrete lining was introduced to insure the permanency of the structure, strengthen it from outward pressure and guard it against injury from accidents which might occur in the tunnel. The side concrete benches were suggested by Mr. Cassatt, President, to confine the trains to the center of the tunnels in case of derailment, and to furnish sidewalks on each side of the trains so as to obviate the necessity of walking on the track.

Refuge niches are constructed in the side benches of the tunnels. Manholes, splicing chambers, pump chambers, and other features for the handling of the electric cables and drainage, are established at intervals.

At points where unusual stresses were anticipated, as for instance where the tunnels pass from rock to soft ground, the shell was composed of steel instead of cast-iron plates. In the North River tunnels the concrete lining in the invert and in the arch was reinforced by longitudinal steel bars, but these were not introduced in the East River tunnels.

Other details connected with the structures, including the drainage, lighting, ventilation, signaling, and electrification systems, will be given in succeeding papers.

Stability of the Sub-River Tunnels.—One of the most important questions connected with the design of these tunnels was their probable stability under the long-continued action of a heavy and rapid railroad traffic. The tunnels are lighter than the materials which they displace even when the weight of the heavy live load is included. In the East River the character of the material seemed to justify the conclusion

that the tunnels would not be displaced even under the action of the live load. In the North River, however, the tunnels are enveloped by a soft silt and it was at first apprehended that some system of supports would be advisable to carry the heavy traffic and insure the tunnels against displacement under its action. To meet this contingency which was then believed to be a very serious one, it was proposed to sink cast-iron screw-piles through the bottom of each tunnel into and through the underlying silt until satisfactory bearing material was reached. The pile supports were worked out in sufficient detail to be embraced in the contract for the construction of these tunnels, with provision, however, for omitting them should it be determined subsequently that their use was undesirable. The contract plans contained provisions for sliding joints where the piles pass through the tunnel floor, so that the live load might be carried directly to the pile heads by a system of girders, and also for attaching the piles directly to the tunnel, the two plans being alternatives.

Investigations, made during the progress of the work to determine the physical character of the silt and its action on the tunnels, suggested the possibility that the use of pile supports might be inadvisable. This view was confirmed by actual experience in the operation of the tunnels of the Hudson Companies between Hoboken, N. J., and Morton Street, Manhattan, which were opened to traffic in February, 1908. The stability of these tunnels under traffic gave further assurance that supports were unnecessary under the North River tunnels of the Pennsylvania Railroad Company, and they were therefore dispensed with.

Cross-Passages Between the Tunnels.—The Bergen Hill tunnels, the land portions of the North River tunnels and the tunnels under Manhattan are connected by cross-passages at intervals varying from 50 to 300 ft. As it was the desire of the Management to provide every arrangement possible to insure the safety of its passengers and employees and also to provide for the convenience of inspection, the question of establishing cross-passages between the tunnels under the rivers was given most careful consideration. The conclusion was finally reached that such passages as it was possible to construct between these tunnels might increase instead of diminish the danger in case of accident. No more cross-passages have therefore been constructed in the sub-river sections, except in the East River, where there is a cross-

passage and pump chamber combined between each pair of tunnels about 750 ft. from the Manhattan bulkhead line.

Probable Results of the Improvements.

In preceding pages reference has been made to the general objects of the improvements included in the project of the Pennsylvania Railroad Company for the New York District. While it is impossible, in this introductory paper, to analyze fully the transportation problem at New York, it seems desirable to indicate briefly some of the more obvious effects which the improvements may be expected to produce upon the distribution and handling of traffic.

New York City owes its position as the business metropolis of the country mainly to its magnificent harbor and the extensive waterfronts on its deep, wide rivers, which furnish unrivaled facilities, at a short distance from the sea, for foreign and domestic water-borne commerce, its foreign commerce being about half the total for the whole country. The water-transportation facilities of the port and its tributaries, therefore, have always been guarded with jealous care, not only by the local commercial interests but also by the General Government.

During recent years, however, the population of the metropolitan district has increased so enormously that New York is now the greatest terminal passenger and freight traffic center in the country; and in manufactures it ranks first among American cities. The new commercial interests thus created are of at least equal importance with those of the water-borne commerce, although their existence and development are largely the result of the water facilities of the port.

The local passenger and freight traffic of the Pennsylvania and of other railroads reaching the west shore of the North River is conducted by car-floats and ferry-boats which deliver their loads at piers on the Manhattan waterfront and elsewhere in the harbor. These boats obstruct and endanger the free navigation of the channels and occupy space along the waterfront greatly needed for the accommodation of the long-distance water-borne commerce, especially on the North River.

In the East River the importance of ferry-boats as a means of traffic distribution has already been greatly reduced by the construction of bridges and tunnels which provide for the greater part of the passenger and vehicular traffic. The North River, however, by reason of its greater width and the comparative slowness of its currents, is by far

the more important waterway for the use of ocean-going vessels of the larger classes. In this river the conditions for the construction of bridges, within the limits of commercial convenience, seem to be practically prohibitory. Tunnels, for the transportation of passengers and the diversion of the freight traffic from the inner waters of the harbor, are apparently the only available means of relief.

When the new line is in operation, a very large part of the New York passenger traffic of the Pennsylvania Railroad will be carried to the New York Station at Seventh Avenue and 33d Street and the rest will go to Cortlandt Street through the Hudson Company's tunnels. Thus a large portion of the Pennsylvania passenger ferry traffic, which amounts to more than 91 000 passengers daily, will be practically eliminated from the water-transportation problem. In addition, a large part of the Long Island Railroad's passengers will use the station at Seventh Avenue and 33d Street, and its ferry traffic will be reduced accordingly.

The new arrangements for the transfer of freight from Greenville to Bay Ridge will relieve the inner waters of the harbor of a large volume of obstructive car-float traffic. There appears to be no reason why this traffic should not be eventually conducted through tunnels under the outer harbor, should future transportation conditions justify the enormous cost of such structures.

It is to be remarked that while these new arrangements greatly reduce the passenger and freight water transportation, they have no effect on the large vehicular traffic across the North River which must continue to be conducted by ferries until it can be otherwise provided for. As long as these conditions exist, ferry-boats must be used in large numbers and continue to obstruct the North River. This difficulty probably cannot be overcome by the construction of bridges, as in the case of the East River, but it does not seem too much to expect that, eventually, tunnels to provide for the vehicular traffic, like the Blackwall tunnel under the Thames, will be established under the North River.

It would be interesting to estimate the increase in railroad traffic capacity resulting from these improvements, but the data required for this purpose are not available. Some idea of the increase in passenger traffic capacity resulting from the establishment of the tunnel line may be obtained by comparing the proposed daily train-movements for

the new station with the train-movements at other important railroad stations. The daily train-movements of six such stations are given in the following table:

	Total trains in and out for 24 hours.	Movement at maximum hour.
Jersey City.....	281	29
Broad Street Station, Philadelphia.	538	48
Union Station, St. Louis.....	462	89
South Terminal Station, Boston...	861	87
Grand Central Station, New York..	357	44
*Pennsylvania Station, New York..	500	50

The freight capacity of the Pennsylvania System at New York has been greatly enlarged by the construction of the Greenville Yard and the facilities connected therewith, but it is impossible to estimate the amount of this increase. However, it is worthy of remark that, during the period from 1900 to 1906, the freight traffic density on the directly-operated lines of the Pennsylvania Railroad Company increased from 3 268 330 to 4 742 081 ton-miles per mile of road, a growth of nearly 50 per cent. Doubtless the improved freight facilities of the New York District had a large influence in the development of this increase.

One of the most interesting points connected with this development of traffic facilities is its influence on the relative distribution of population in the different parts of the metropolitan district. In 1907 the population per acre of the different divisions of Greater New York was reported as follows: Manhattan, 157; Brooklyn, 29; Bronx, 14; Queens, 3; Richmond, 2. The effect of new lines connecting some of these districts, and sections of New Jersey not far from the North River, with the business center of the city will undoubtedly be to increase greatly their population-density. It does not seem probable that the population-density of Manhattan will be sensibly reduced by these improvements, for they stimulate the increase of population, and apparently no increase of transportation facilities can keep up with the growth of the city. The population of a great commercial city must be congested near the business center. This is a necessary condition of its existence. All that can be done to meet this condition is to

* Proposed train service when station is opened, the ultimate capacity of the Station being in excess of 1 000 trains per day.

provide all possible facilities for moving the people into and out of the business districts and within its limits.

During recent years the business population of the lower part of the Borough of Manhattan has become greatly congested. Very high buildings, providing business accommodations for large numbers of people, have been constructed, and these people must move to and from their working places at about the same times, that is, at the "rush hours" in the morning and afternoon, at the beginning and ending of the working day. Every effort has been made to provide for this immense and rapidly increasing local passenger traffic, by the construction of surface, elevated, and subterranean railways; but the demand for transportation has increased much faster than the facilities can be provided, and it is evident that the limit of down-town passenger traffic facilities has been very nearly reached.

Apparently, the only remedy for these conditions is the movement of business and the people transacting it up-town or to the Boroughs of Brooklyn and Queens, which are now readily accessible by tunnels and subways. This movement, of course, is resisted by the great real estate and money investments centered in the lower part of the city, but, notwithstanding this resistance, the improvement has commenced and has rapidly advanced. The great retail houses are being established above 23d Street; the banks and brokers' offices are rapidly appearing around the new business center of the city. The facilities afforded by the telephone and the subway for communication with the money center have doubtless greatly promoted this up-town movement.

When the Pennsylvania Tunnel Extension is in operation, the easiest and quickest way for the passenger to reach the city from Newark will bring him into the Pennsylvania Station at Seventh Avenue and 33d Street. The schedule fast time from Newark to the New York Cortlandt Street Station is now 25 min. This may be reduced to about 18 min. by the use of the Hudson Company's tunnels, and while this involves inconvenience in changing transportation at Jersey City, yet it brings the traveler three blocks nearer Broadway. The time from Newark to the Pennsylvania Station will be about 17 min., and the trip will be made without change of transportation, so that, undoubtedly, by far the greater part of the Pennsylvania's passenger traffic desiring to reach the shopping and hotel center of the city will go to the new up-town station.

The effect of the Tunnel Extension in increasing the volume and rapidity of the up-town movement and the real estate values will be very great; indeed, its influence is already apparent, although the line is not yet opened for traffic. With the extension of the present subway down town on the west side with direct connections to Brooklyn, and up town from 42d Street to the Bronx, with connections to permit convenient transfers between these two straightaway subways—one on the east side and the other on the west side of Manhattan—will make the Pennsylvania Station a great center for receiving and distributing passenger traffic between all the Boroughs of the City and outlying points. The new post office to be established adjacent to the Terminal Station will also greatly assist in accelerating the up-town movement.

In concluding this account of the New York Tunnel Extension project, the writer desires to pay a tribute of admiration and respect to the memory of the late A. J. Cassatt, President of the Pennsylvania Railroad Company, to whom the conception, design, and execution of the project are mainly due. His education and experience as a civil engineer, his thorough knowledge of all the details of railroad construction, operation, and management, gained by long and varied service, the directness, clearness, and strength of his mind, and his great executive ability, placed him at the head of the railroad men of the country. In the consideration of great problems, whether of transportation, finance, commerce, or political economy, he was almost unequaled, owing to the breadth, originality, and decisiveness of his character; yet his manner to his subordinates was so direct and simple that he seemed unconscious of his own superiority. Great as it is, the New York plan of improvement is only one item in a far-reaching scheme of development which became the policy of the Pennsylvania Railroad Company through Mr. Cassatt's advice and influence, yet his strongest interest was doubtless centered in the New York works. It is the sincere regret of all connected with the design and execution of the project that he did not live to see its completion.

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AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

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THE NEW YORK TUNNEL EXTENSION OF THE PENNSYLVANIA RAILROAD. THE EAST RIVER DIVISION.*

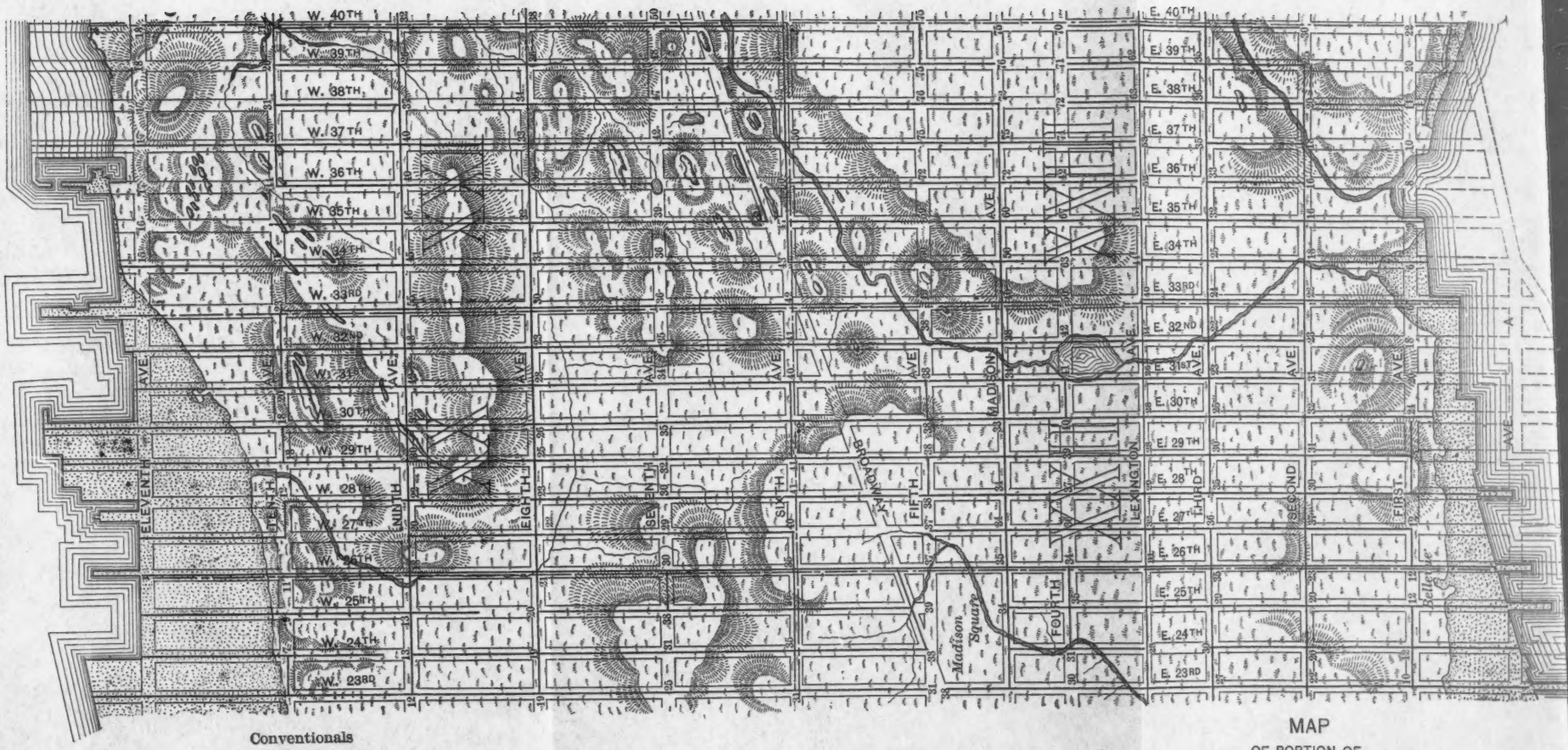
By Alfred Noble, Past-President, Am. Soc. C. E.

A general outline of the work included in this Division has been given by General C. W. Raymond, M. Am. Soc. C. E., in the first paper of the series. The few pages following are intended only as a note to connect his paper with the more detailed descriptions of the execution of the work which follow.

Soon after the Company's project was made public, in the latter part of 1901, borings were begun in the East River, and a few weeks later in Manhattan and Long Island City. A preliminary base line was measured on the Manhattan side, and temporary transit stations were established on buildings from which all borings in the river were located. The river borings were all wash-borings made from a pile-driver boat. After the results were plotted on the map, contour lines were drawn to indicate the rock surface, and profiles along the tunnel lines were plotted from the contours; as the borings were preliminary to the final location of the tunnels, and in many cases at some distance from the tunnel lines, considerable divergence from the actual rock surface was expected, and realized in a few places, yet on the whole the agreement was very good. The borings revealed two depressions

* This paper and that of Gen. Raymond are printed as introductions to others describing the New York Tunnels and Terminal of the Pennsylvania Railroad, which will appear in the October, 1909, and subsequent numbers of *Proceedings*.

PLATE
PAPERS, AM.
SEPTEMBE
NOBLE
EAST RIVER D VISION : PENN



Conventionals

Made Land

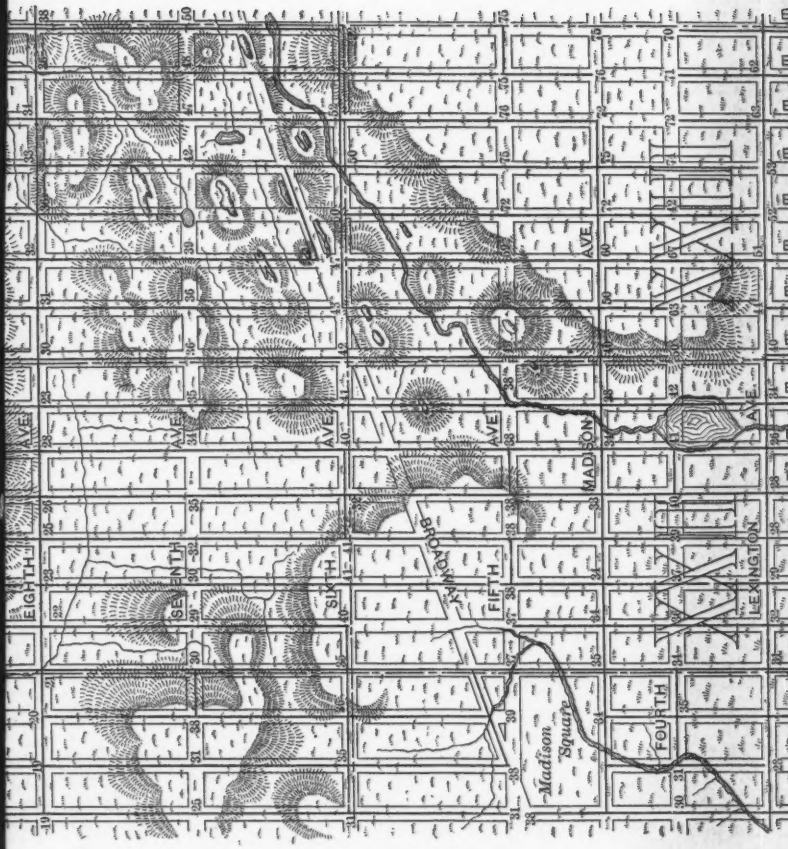
Meadow Land

Sewers

SCALE OF FEET

0 500 1000 2000

MAP
OF PORTION OF
MANHATTAN ISLAND
FROM 23D TO 40TH STREETS,
SHOWING FORMER TOPOGRAPHY
FROM MAP MADE BY
GEN. EGBERT L. VIELE
IN
1865



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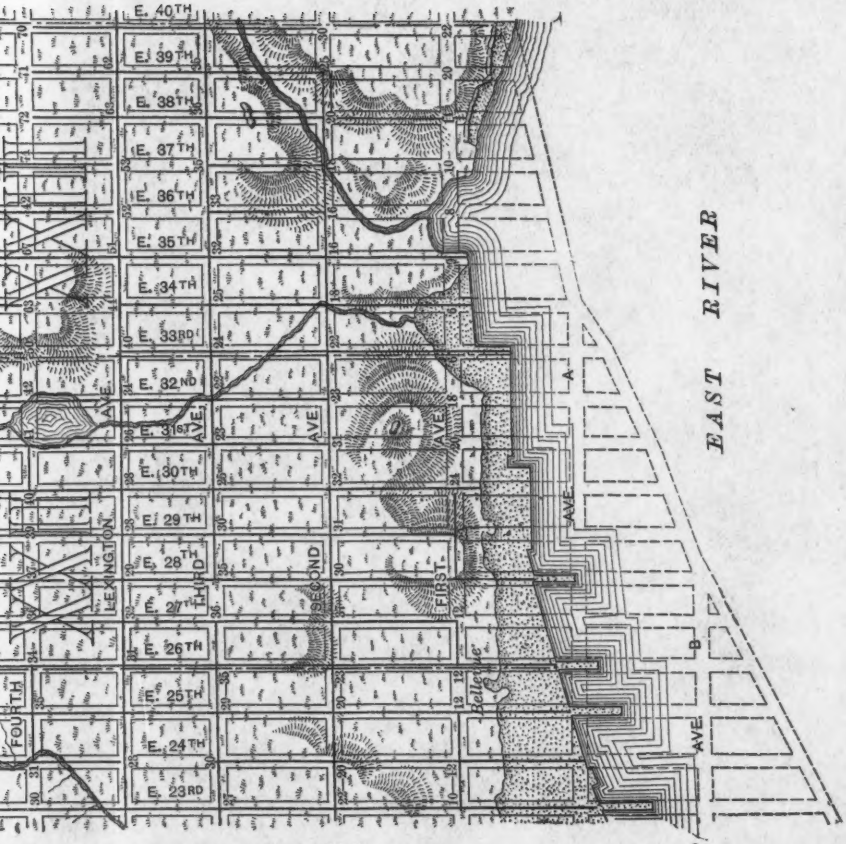


MAP
OF PORTION OF
MANHATTAN ISLAND
FROM 23D TO 40TH STREETS,
SHOWING FORMER TOPOGRAPHY
FROM MAP MADE BY
GEN. EGBERT L. VIELE
IN
1865

UNIV. OF MICH.

or channels where the rock surface passed below the grade of the projected tunnels, these depressions being separated by a rock reef which extends down stream from Blackwell's Island. In 32d and 33d Streets in Manhattan, borings were made from the river to the station site at intervals of about 100 ft., wash-borings and core-borings alternating. In Long Island City, where the tunnel lines were to pass diagonally under the passenger station building and passenger yard of the Long Island Railroad and under streets and private property, the arrangement of borings was less regular, although the alternation of wash-borings and core-borings was carried out as far as practicable. After the final location of the work, additional borings were made, particularly on shaft sites and also along the approaches and in the Sunnyside Yard, Long Island City.

A triangulation was carried across the river with a measured base on each side. It was impossible to measure directly between the extremities of either base. The bases were measured with 100-ft. steel tapes, supported every 20 ft., stretched with a uniform pull, and frequently compared with standardized tapes. On account of the crowded condition of the streets during the hours of daylight and evening, most of the work was done between 10 P. M. and 5 A. M. Similar measurements were made in the streets along the tunnel lines. Angle readings were repeated many times, as is usual in such work. Fig. 1 shows the triangulation, the street measurements being omitted.

Levels were first transmitted across the river by simultaneous observations of the river surface; then by several repetitions, across Blackwell's Island and the narrow channels on each side, where the longest sights were about 1 100 ft.; and, finally, by several lines through the tunnel of the East River Gas Company at 71st Street.

The franchise granted by the City of New York provided for the sale to the Railroad Company of the portions of 32d Street between Seventh and Eighth Avenues, and between Eighth and Ninth Avenues. Later, the Company acquired by purchase the portion of 32d Street between Ninth and Tenth Avenues. The franchise granted sub-surface rights under streets around the station site to within 19 ft. of the street surface under Seventh, Eighth, and Ninth Avenues; to within 30 in. of the street surface under 31st and 33d Streets, except that, under the sidewalks opposite the station, that is to say, the south sidewalk in 31st Street and the north sidewalk in 33d Street, the con-

struction must be at least 5 ft. below the street surface. In carrying out the work, full use of these rights was made under Eighth Avenue, but only under such portions of Seventh and Ninth Avenues as were indispensable for access by trains to the station area. It was not practicable to make full use of the rights granted under 31st and 33d Streets without incurring great expense for supporting adjacent build-

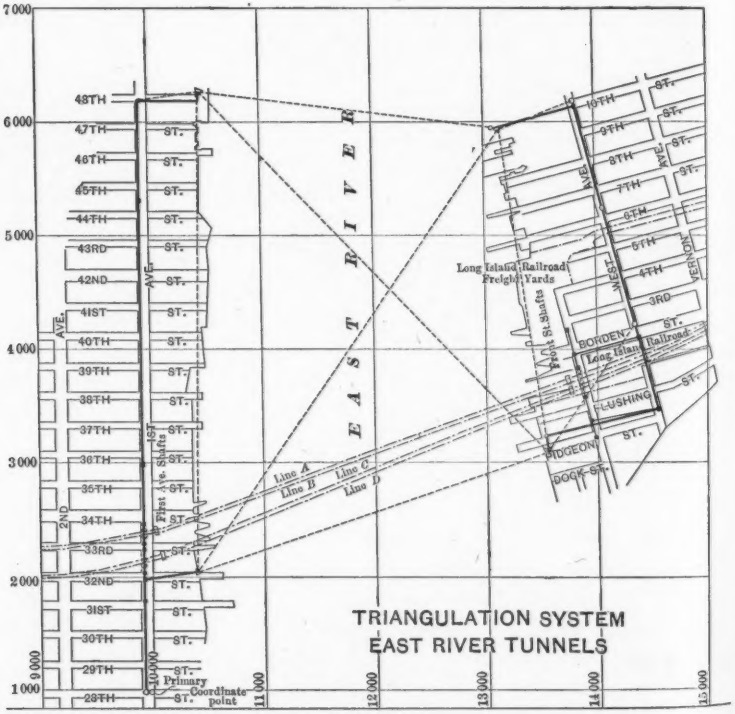
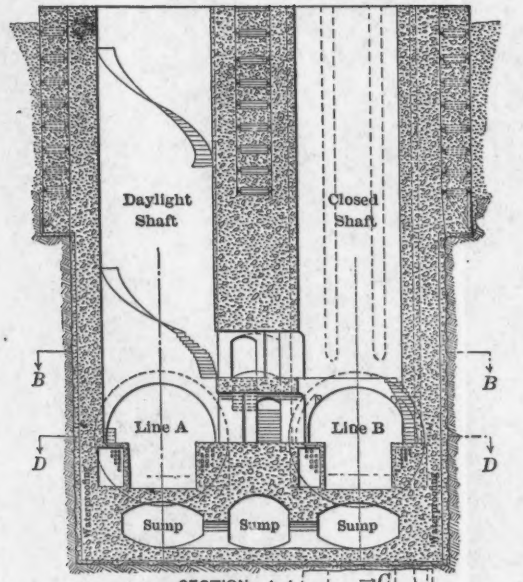


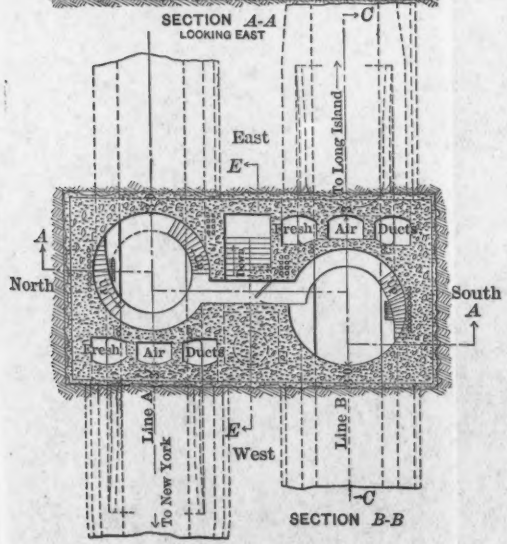
FIG. 1.

ings or for injuries to them, and, after careful consideration, the arrangement shown in the plans was decided on, making about 45% of the sub-surface area under these streets available at track level.

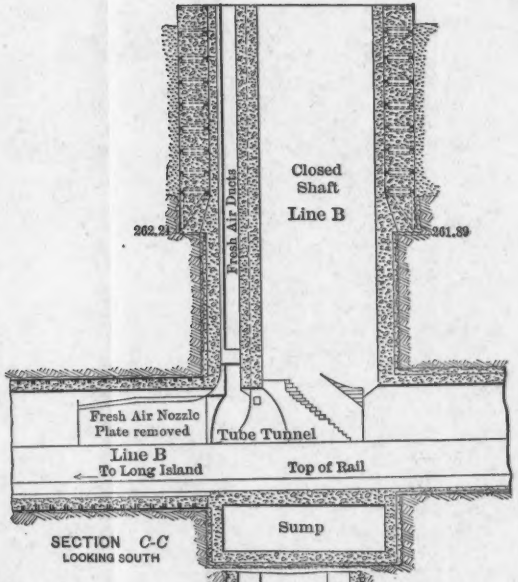
The work of the East River Division at this site embraced the excavation to the depth necessary for railroad tracks, and the building of a retaining wall extending in 31st Street from the east side of Ninth Avenue to the west side of Seventh Avenue, thence northward along Seventh Avenue for a distance of 155.5 ft.; also a retaining wall



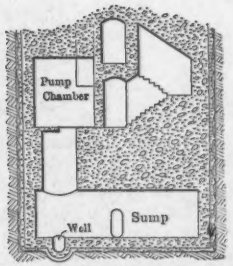
SECTION *A-A*
LOOKING EAST



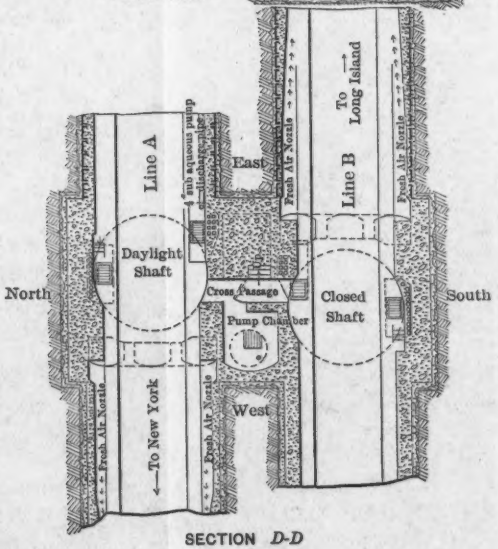
SECTION *B-B*



SECTION *C-C*
LOOKING SOUTH



SECTION *E-E*
LOOKING NORTH



SECTION *D-D*

MANHATTAN SHAFT
LINES *A* AND *B*

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in 33d Street from the west side of Seventh Avenue to the east side of Ninth Avenue, and thence southward along Ninth Avenue for a distance of 136.3 ft. This work was placed under contract June 21st, 1904, with the New York Contracting and Trucking Company, and later assigned by that company to the New York Contracting Company—Pennsylvania Terminal, and was carried out under the direction of George C. Clarke, M. Am. Soc. C. E., as Resident Engineer, by whom it will be described in detail.

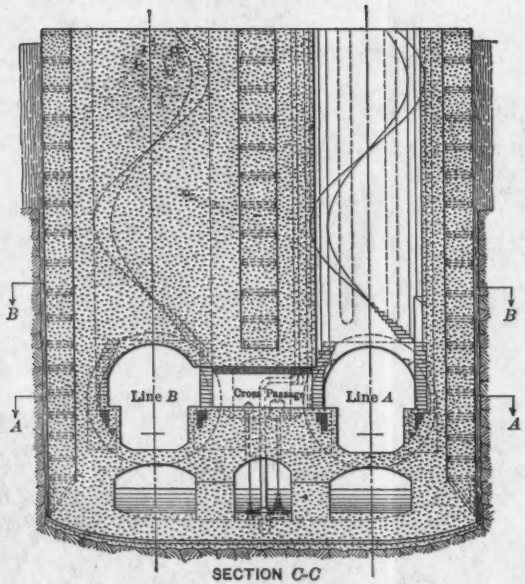
The station tracks leading eastward from the station will converge under Seventh Avenue and for some distance farther east, and pass into two three-track tunnels, one under 32d Street and the other under 33d Street, at the respective distances of 192 and 402 ft. from Seventh Avenue. A typical cross-section of the three-track tunnel is shown on Plate LXIV. The converging sections were considered as easterly extensions of the station, and were not included in the East River Division. Within a few hundred feet (Plate LXVI), the tracks are reduced to two, each passing into a single tube, the two tunnels under each street being formed in one excavation, the distance between center lines of tunnels being 20 ft. 4 in. This construction has been termed a twin tunnel, and a typical cross-section is shown on Plate LXIV. The tunnels continue on tangents under the streets to Second Avenue where they curve to the left by 1° 30′ curves, passing under private property, gradually diverging and passing through shafts just east of First Avenue. About 350 ft. west of the shaft, the divergence of the two lines from each street becomes sufficient to leave a rock dividing wall between them, and thence eastward each tunnel is formed in a separate excavation. A typical cross-section of the two separated tunnels is shown on Plate LXIV.

It thus appears that eastward from the station the lines constitute a four-track railroad, each track being in a separate tunnel; for convenience of the work these lines were designated *A*, *B*, *C*, and *D*, from north to south.

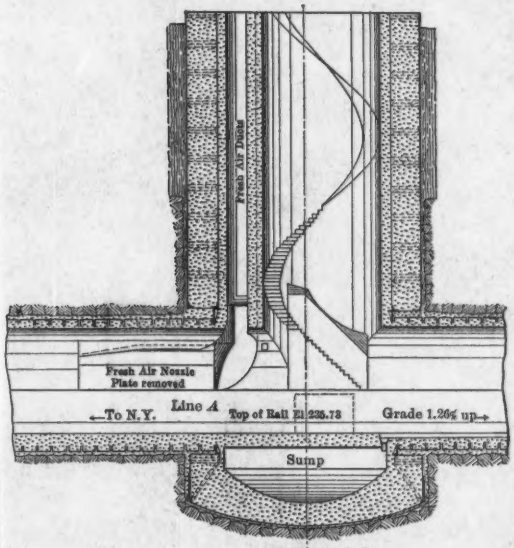
At an early date, when the organization of the engineering staff was taken up, Charles L. Harrison, M. Am. Soc. C. E., was appointed Principal Assistant Engineer. He was directly in charge of all parts of the work, and all Resident Engineers reported to him. George Leighton, M. Am. Soc. C. E., was placed in charge as Resident Engineer of the 33d Street lines from the west end of the three-track tun-

nel to the shaft and also eastward from the shaft under East River. As he was not then able to endure the effects of compressed air, the work under the river was transferred to James H. Brace, M. Am. Soc. C. E., as Resident Engineer. Before the completion of the land tunnels under 33d Street, Mr. Leighton accepted more responsible employment elsewhere, and Mr. Brace assumed charge of them also. Francis Mason, M. Am. Soc. C. E., was in charge as Resident Engineer of the 32d Street lines during their entire construction, and also of the tunnels extending these lines eastward from the First Avenue shaft under the river.

The work just described as the 32d and 33d Street lines, terminating at the easterly end at the First Avenue shafts, was placed under contract on May 29th, 1905, with the United Engineering and Contracting Company. The plans then provided for three-track tunnels from the west end of the work under the contract eastward 1 628 ft. in 32d Street and 1 418 ft. in 33d Street to the west line of Fifth Avenue, with a descending grade of 0.4%; this was to constitute, in a degree, an extension of the station, where trains could stand without brakes while awaiting signals to proceed to or from the station. From Fifth Avenue eastward to the lowest point under the river, the grade was to be 1.5% on all lines. Later, during construction, when excavating westward under 33d Street from Fifth Avenue, the surface of the rock was broken through, disclosing quicksand; within the next few days trial drill holes through the tunnel roof at 32d Street and Fifth Avenue showed a thin cover with quicksand above it. The conditions had been indicated in a general way by borings made before construction was begun, but they proved to be rather worse than anticipated. On the topographical map of Manhattan Island, made by General Egbert L. Viele in 1865, is shown a watercourse which had its source near what is now Broadway and 44th Street, flowing thence along the west side and south end of Murray Hill, passing under the present site of the Waldorf-Astoria Hotel, crossing 33d Street at the point where the rock surface was broken through in the tunnel excavation, as above stated, crossing 32d Street at its intersection with Fifth Avenue, where trial drilling showed thin rock cover over the tunnel excavation, passing thence eastward a short distance south of 32d Street, which it recrossed near Third Avenue, and finally discharging into the East River near 34th Street, and a little west of the present First Avenue. The

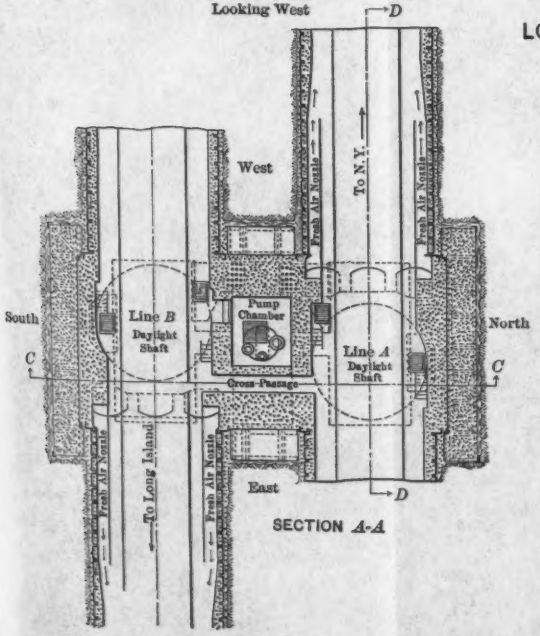


SECTION *C-C*
Looking West

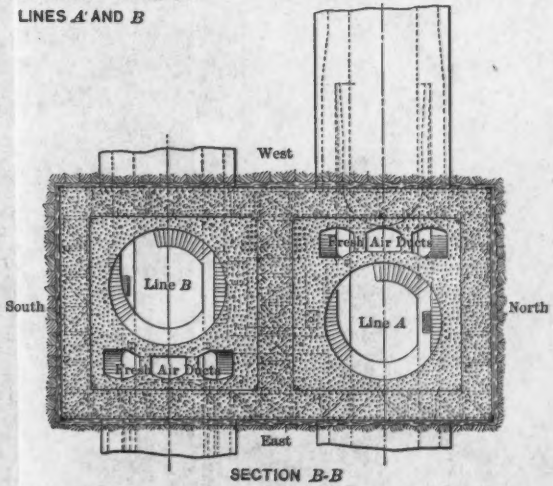


SECTION *D-D*
Looking North

LONG ISLAND SHAFT
LINES *A* AND *B*



SECTION *A-A*



SECTION *B-B*

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ancient creek apparently followed the course of a valley in the rock, the valley having become filled to a considerable depth with very fine quicksand. This concurrence of depressions in the rock surface with the watercourse shown on Viele's map was noted in so many places and the difficulties of construction were so serious at these places, that a section of the map showing the old topography along and adjacent to the station and tunnel lines is reproduced in Plate LXI.

The unfavorable conditions developed at Fifth Avenue affected both the construction of the tunnels and the maintenance of adjacent buildings. It would be necessary to construct the tunnels in open cut for a large part of the way westward, causing serious inconvenience to the public; the buildings were mostly of the older class, founded in earth, but there were several modern high buildings with foundations in the same material; some of these had been built since the tunnels were planned. In view of these added risks and the increased cost of construction, the value of the three-track construction was reconsidered, and two important changes were made in the plans. The first of these was to continue the twin tunnel westward to Sixth Avenue in 32d Street, and to a point 180 ft. west of Sixth Avenue in 33d Street; the twin tunnel being 9½ ft. less in height than the three-track tunnel and 9 ft. narrower, the change reduced the difficulties considerably. Where the three-track tunnel was thus eliminated, there was no longer objection to a steeper grade, so that, going eastward from the station, a grade of 0.8% in 33d Street and 0.9% in 32d Street was substituted for the original 0.4% grade. From the west line of Fifth Avenue eastward short sections with descending grades of 0.3% connect with the original 1.5% grade near Madison Avenue. The effect of these two changes—type of tunnel and grade—was to lower the roof of the tunnels at Fifth Avenue about 15 ft., which made it practicable to avoid open cutting east of Sixth Avenue.

A full account of the construction of the cross-town tunnels will be given by the Resident Engineers.

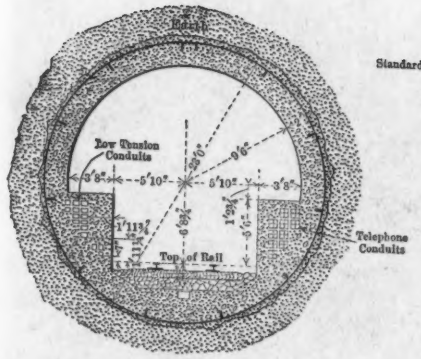
Permanent shafts were made on both sides of the East River, those in Manhattan being located a few feet east of First Avenue, and those in Long Island City being located, one in the so-called Annex Slip, the other in the pier just south of it. The two railroad lines coming from 32d Street in Manhattan, and curving to the left at Second Avenue, are about 34 ft. apart between centers at First Avenue, and

it was convenient to make the shaft large enough to cover both lines. Borings had shown that the excavation for the tunnels would break out of the rock about 200 ft. east of First Avenue. It was desirable to carry the tunnel excavation eastward from the shaft in normal air far enough to permit of building at least 50 ft. of tunnel and installing air-locks, so that compressed air might be available when the rock surface was broken through. The location adopted, and shown on Plate LXV, had the further advantages that the rock surface was several feet above the level of the top of the tunnels, and access to the river for receiving and discharging materials could be had without crossing any street. Similar reasons governed the location of the north shaft for the lines from 33d Street. On the Long Island side of the river there were only two feasible locations meeting these conditions, particularly in respect to a safe thickness of rock above the tunnels, one near the pierhead line, the other just outside the bulkhead line, and for many minor reasons the latter was preferable. The center lines of each pair of tunnels were 37 ft. apart, and each shaft, therefore, was made to cross both lines of a pair, the same as on Manhattan side of the river. It was not expected, however, that the Long Island shafts could be built conveniently or the tunnels begun from them in normal air.

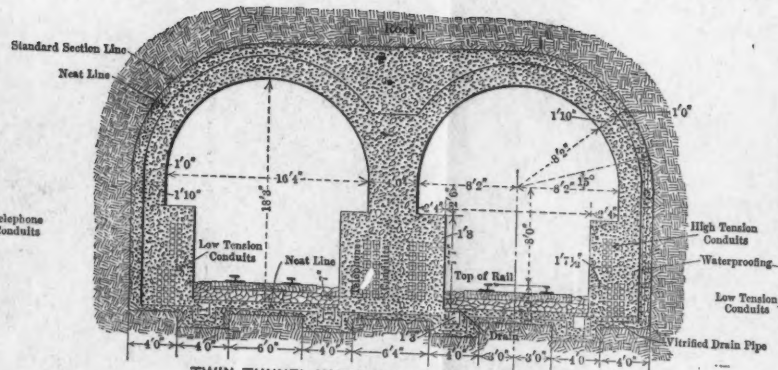
The decision to make the shafts of permanent construction was based not only on the desirability of having access to and egress from the tunnels near the banks of the river for convenience of the workmen or exit for passengers in case of accident, but to facilitate ventilation; these locations divide the entire lengths of tunnels east of the station into three parts, two of which were approximately 4 000 ft. each, and the other about 5 500 ft. The accident risk was believed to be very small, while much weight was given to the feature of facilitating ventilation. Further studies have enhanced the importance attached to ventilation, and it is now intended to provide appliances for mechanical ventilation at all shafts. The plans of the shafts are shown on Plates LXII and LXIII. The caissons for the shafts are of structural steel, with double walls, filled between with concrete, including a cross-wall between and parallel to the tunnels. All these structures were fitted for sinking with compressed air, if that should prove necessary.

Although borings had shown that rock would be found at all the

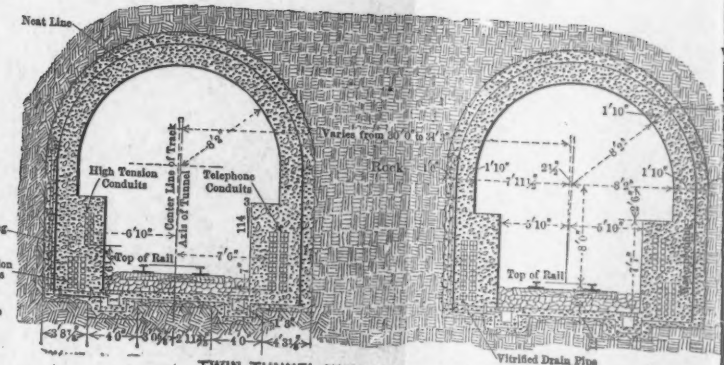
TYPICAL TUNNEL SECTIONS



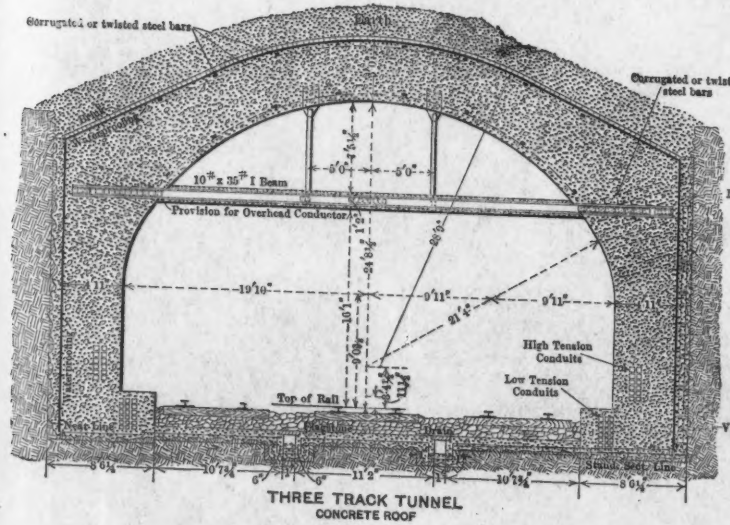
TUBE TUNNEL



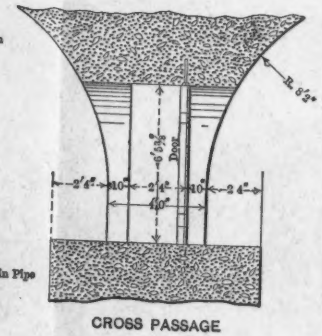
TWIN TUNNEL WITH CONCRETE-DIVIDING WALL



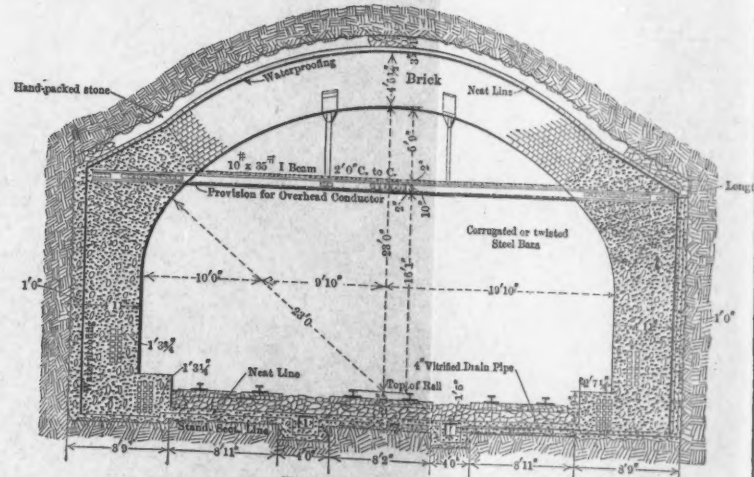
TWIN TUNNEL WITH ROCK-DIVIDING WALL



THREE TRACK TUNNEL
CONCRETE ROOF

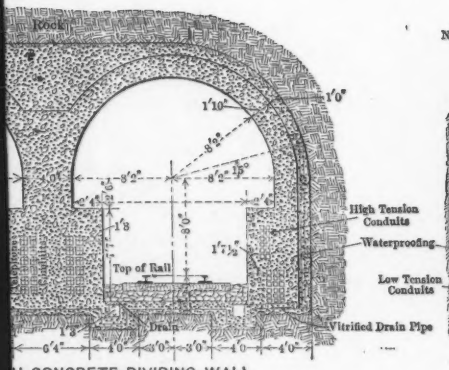


CROSS PASSAGE

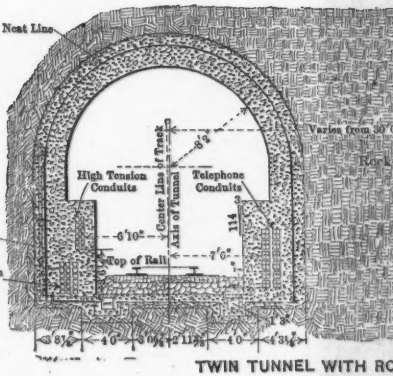


THREE TRACK TUNNEL
BRICK ROOF

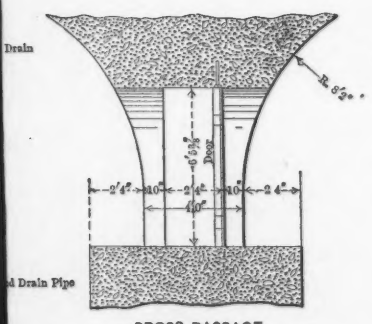
TYPICAL TUNNEL SECTIONS



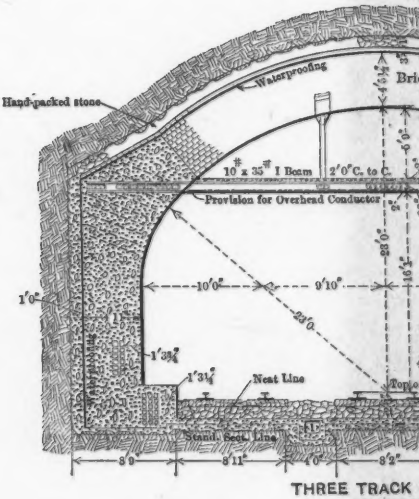
H CONCRETE-DIVIDING WALL



TWIN TUNNEL WITH RC

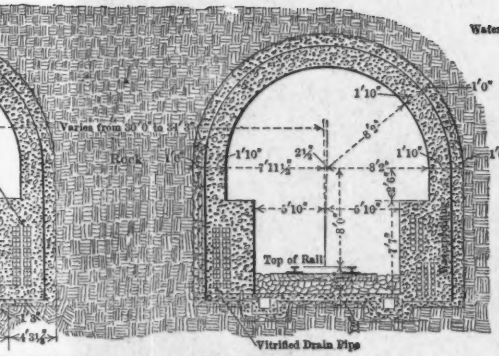


CROSS PASSAGE

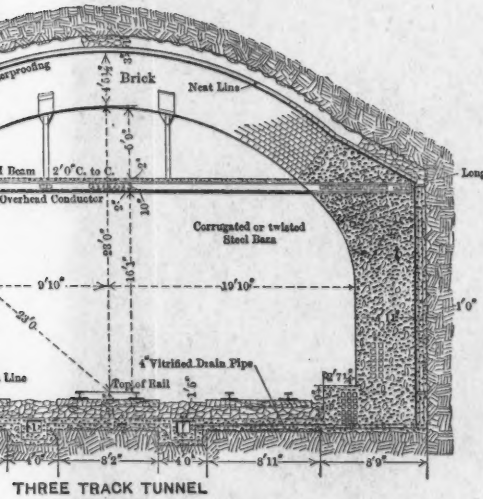


THREE TRACK
BRICK ROO

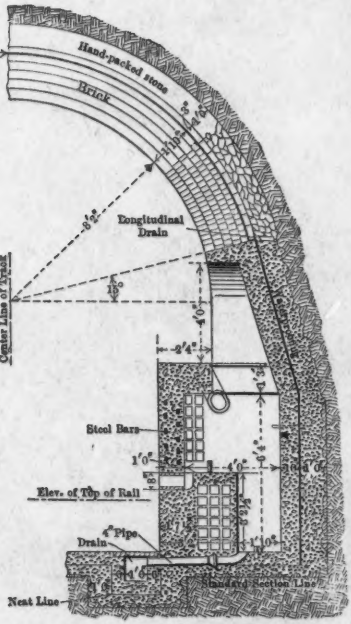
PLATE LXIV.
PAPERS, AM. SOC. C. E.
SEPTEMBER, 1909.
NOBLE ON
EAST RIVER DIVISION: PENNSYLVANIA R. R. TUNNELS.



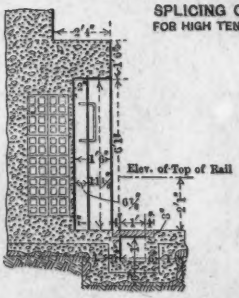
TUNNEL WITH ROCK-DIVIDING WALL



THREE TRACK TUNNEL
BRICK ROOF



SPLICING CHAMBER
FOR HIGH TENSION CABLES



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shaft sites several feet above the tunnel level, it could not be determined in advance of excavation whether the caissons would have to be sunk to full depth; if sound, unfissured rock were found, the sinking could be stopped above the tunnel level; but, if not, the caissons, in any case, would have to be sunk far enough to permit placing a water-tight floor below the tunnels, and the tunnels themselves begun through openings in the side-walls of the caisson; such openings, therefore, closed by removable bulkheads, were provided in all caissons.

As already stated, the grade of 1.5% from Fifth Avenue eastward was fixed with reference to the lowest point of the river bed in order to give the requisite cover over the tunnels at the deepest point of the channel on the west side of the reef, where the river bottom was about 60 ft. below mean high tide for a short distance. On the other hand, as the use of compressed air in building the tunnels was anticipated, an excessive depth below the water surface was to be avoided as far as possible; it was necessary, however, to continue the descending grade some further distance until the tunnels were mostly in rock, so that drainage sumps under the tunnels could be made readily. Eastward from the sumps the tunnels had a rising grade of 0.7% to the established bulkhead line on the Long Island side, giving a cover at the points where the tunnels enter rock, a short distance westward, of about 10 ft. (if the dredging plane should be fixed at some future time at 40 ft. below mean low tide, as may be reasonably anticipated). Eastward from the bulkhead line, Tunnels *A*, *B*, and *D* have ascending grades of about 1.25%, while Tunnel *C* rises at the rate of 1.9% in order to effect a crossing over Tunnel *B* west of the portals. This feature was introduced in order to place the two west-bound tracks together through the Sunnyside Yard, and the heavier grade, being downward with the traffic, was not objectionable.

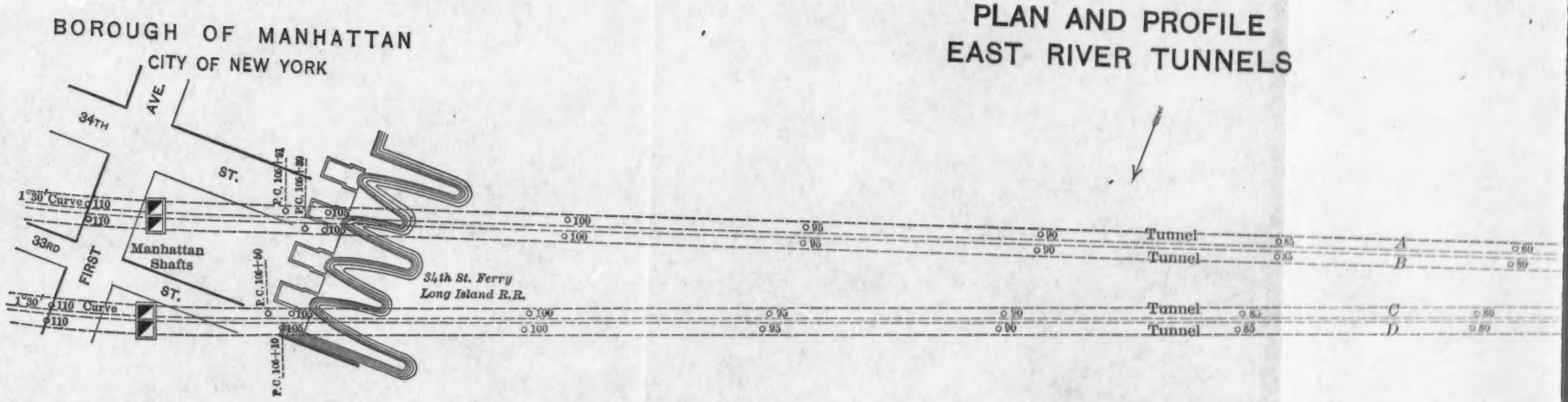
The arrangement of grades and tracks in the approaches and in Sunnyside Yard would require the introduction of too much detail to be taken up here, but will be dealt with in the paper on the Sunnyside Yard.

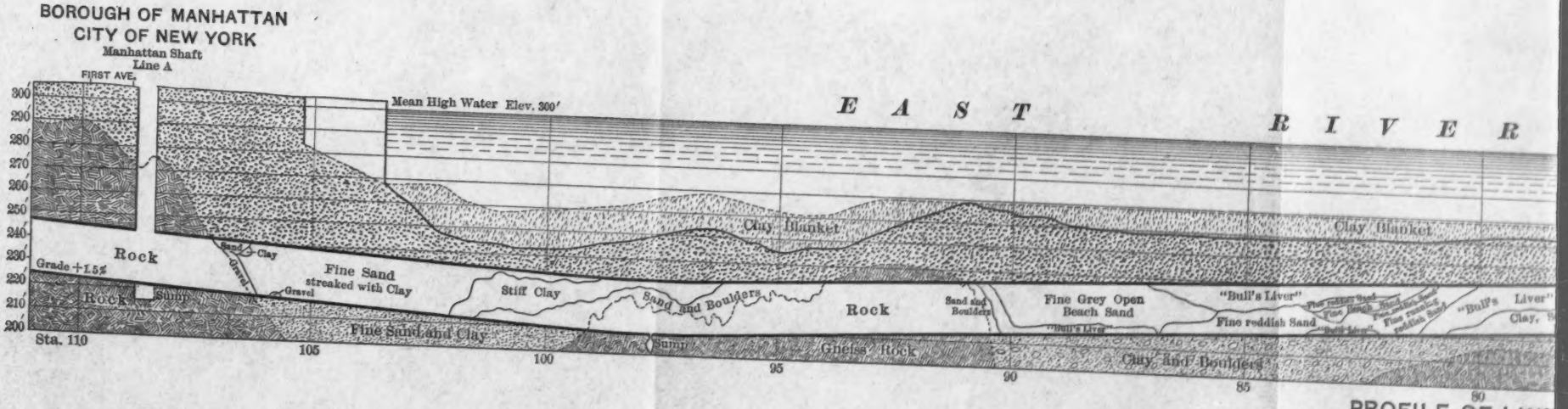
It was recognized from the inception of the project that the tunnels under the East River would be the most difficult and expensive section of the East River Division. The borings had shown a great variety of materials to be passed through, embracing quicksand, coarse sand, gravel, boulders, and bed-rock, as well as some clayey materials. (See

Plate LXV.) The rock was usually covered by a few feet of sand, gravel, and boulders intermixed, but, in some places, where the rock surface was at some distance below the tunnel grade, the material met in tunneling was all quicksand; the nearest parallels in work previously done were some of the tunnels under the Thames, particularly the Blackwall tunnel, where open gravel was passed through. Before the plans for the East River tunnels were completed, work had been resumed, after many years' interruption, in the old Hudson River tunnels between 15th Street, Jersey City, and Morton Street, Manhattan, and sand materials were passed through for a short distance. These experiences satisfied nearly all the engineers in any way connected with the work that the shield method was the most suitable for the East River tunnels, and the plans for the work were based on its adoption. (See Plate LXIV for cross-sections, etc.) Other methods, as stated by General Raymond in the introductory paper, were advocated, particularly caisson constructions and the freezing process, the latter being urged very strongly, and, when proposals were invited, in October, 1903, bidders were informed that alternative methods would be taken into consideration.

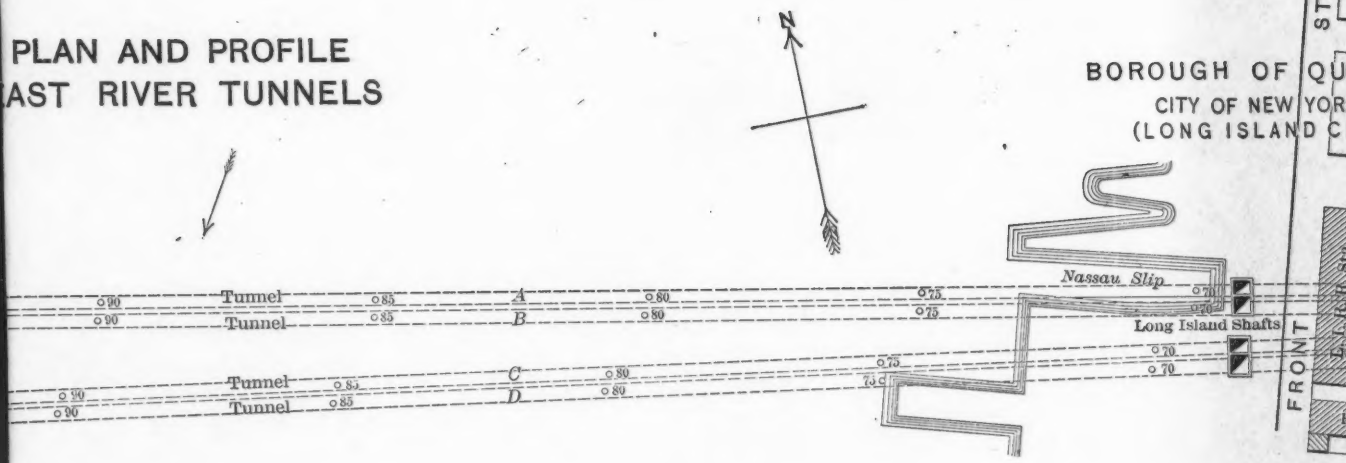
Bids were received and opened on December 15th, 1903. Only one bidder proposed to carry out the work on the basis of unit prices, but the prices were so low that the acceptance of the proposal was deemed inadmissible; no bid based on caisson methods was received; several offers were made to perform the work by the shield method, in accordance with the plans, for a percentage of its cost, and one was submitted, on a similar basis, covering the use of the freezing method. The firm of S. Pearson and Son, Limited, of London, England, submitted a proposal for building the tunnels by the shield method, on a modification of the percentage basis, and as this firm had built the Blackwall tunnel within the estimates of cost and was the only bidder having such an experience and record in work in any way similar to the East River tunnels, negotiations were continued between that firm and the railroad company.

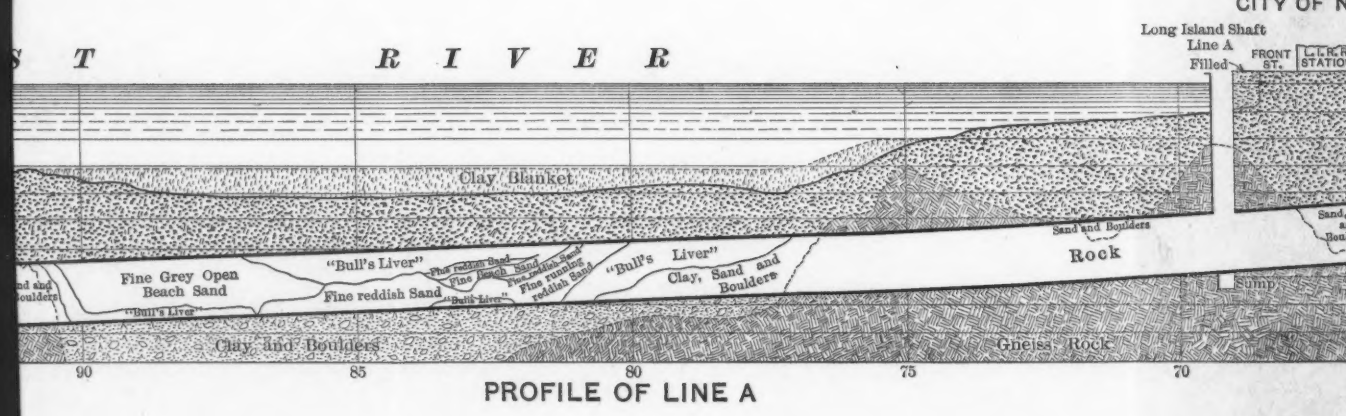
The original plans and specifications contemplated that all tunnels between the First Avenue shafts in Manhattan and East Avenue in Long Island City would be shield-driven, and that work would proceed simultaneously eastward from the First Avenue shafts and both eastward and westward from the Long Island City shafts located west of


BOROUGH OF MANHATTAN
CITY OF NEW YORK
PLAN AND PROFILE
EAST RIVER TUNNELS
34TH
AVE.
ST.
33RD
FIRST
Manhattan Shafts
34th St. Ferry
Long Island R.R.
1°30' Curve
Tunnel
A
B
C
D


BOROUGH OF MANHATTAN
CITY OF NEW YORK
Manhattan Shaft
Line A
FIRST AVE.
Mean High Water Elev. 300'
EAST RIVER
Rock
Grade +1.5%
Sump
Sand & Clay
Gravel
Fine Sand streaked with Clay
Stiff Clay
Sand and Boulders
Clay Blanket
Fine Grey Open Beach Sand
"Bull's Liver"
Fine reddish Sand
Fine Sand and Clay
Gneiss Rock
Clay and Boulders
Sta. 110
105
100
95
90
85
80
PROFILE OF LINE

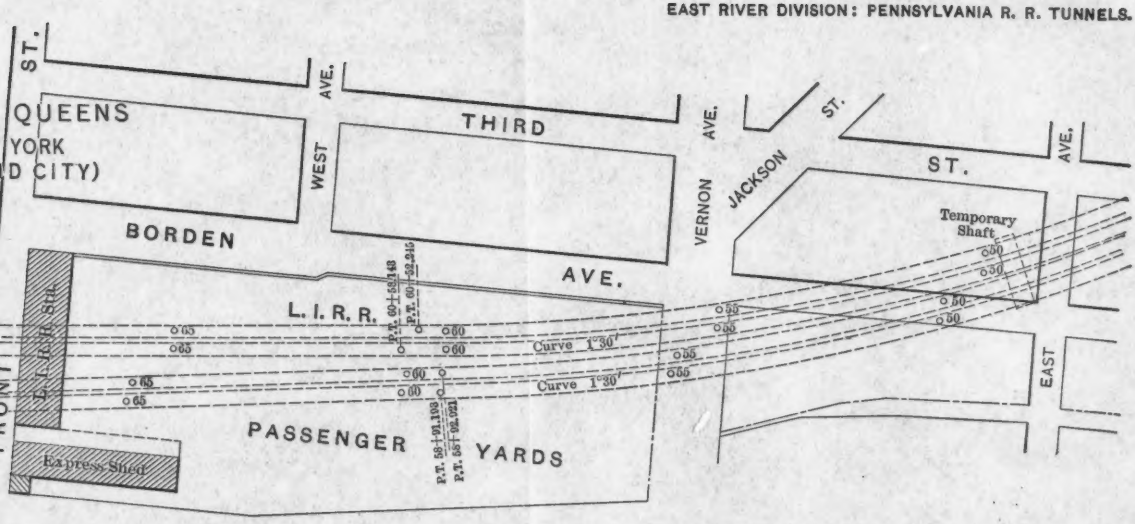
PLAN AND PROFILE
AST RIVER TUNNELS



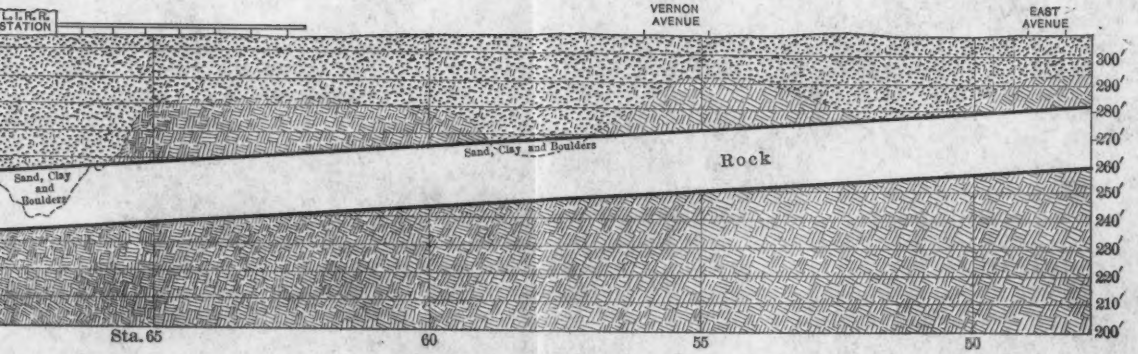


PROFILE OF LINE A

PLATE LXV.
PAPERS, AM. SOC. C. E.
SEPTEMBER, 1909.
NOBLE ON
EAST RIVER DIVISION: PENNSYLVANIA R. R. TUNNELS.



UGH OF QUEENS
OF NEW YORK



UNIV. OF MICH.

Front Street at the river, requiring twelve shields. When making their proposal, S. Pearson and Son, Limited, suggested that shields might be started from the east end of the work and arrive at the Front Street shafts as soon as these shafts could be completed, and proposed sinking a temporary shaft transversely across all four lines near the east end of the work just west of East Avenue, from which, within a short time, to drive toward Front Street by the use of shields. The railroad company accepted the suggestion for the additional shaft, although the greater part of the tunnels east of Front Street was built without shields. After several months of negotiation, a contract was entered into on July 7th, 1904, with S. Pearson and Son, Incorporated, a corporation of the State of New York organized by the English firm for the purpose of entering into and carrying out this contract. The main features had been agreed upon, and work had begun about two months before. The contract embraced the permanent shafts in Manhattan and Long Island City, the tunnels between these shafts, and their extension eastward in Long Island City to East Avenue, including in all about 23 600 ft. of single-track tunnels. The contract had novel features, and seemed to be peculiarly suitable for the unknown risks and the unusual magnitude of the work. A fixed amount was named as contractor's profit. If the actual cost of the work when completed, including this sum named as contractor's profit, should be less than a certain estimated amount named in the contract, the contractor should have one-half of the saving. If, on the other hand, the actual cost of the completed work, including the fixed sum for contractor's profit, should exceed the estimated cost named in the contract, the contractor should pay one-half the excess and the railroad company the other half; the contractor's liability was limited, however, to the amount named for profit plus $1 000 000; or, in other words, his maximum money loss would be $1 000 000. Any further excess of cost was to be borne wholly by the railroad company. The management of the work, with some unimportant restrictions, was placed with the contractor; the relations of the engineer, as to plans, inspection, etc., were the same as in ordinary work, and the interest of the contractor to reduce cost was the same in kind as in ordinary work.

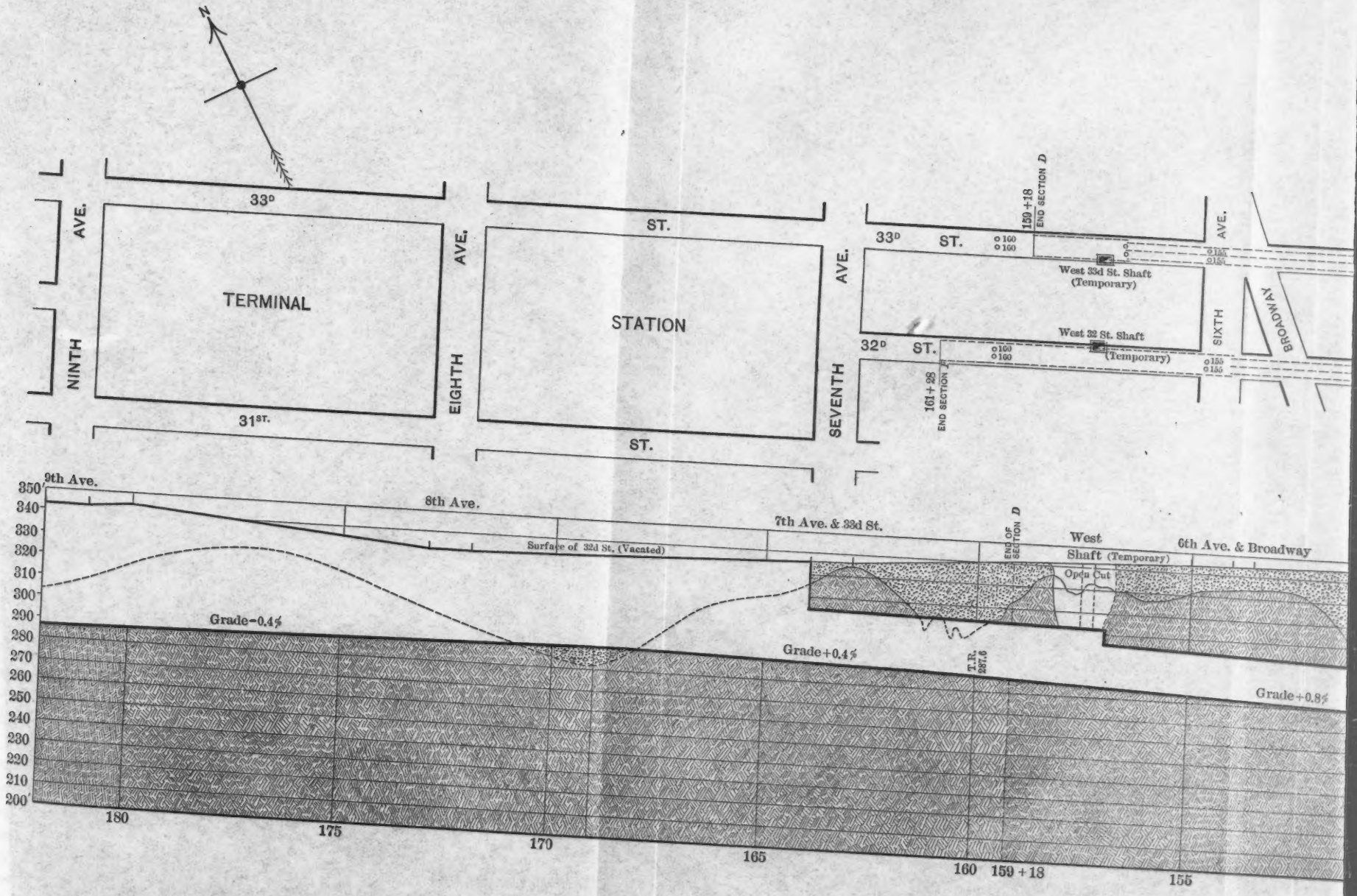
On account of the extent of the work embraced in this contract, and the dangerous exposure to compressed air required in most of it, it was divided into three residencies; two of these, including also the

cross-town tunnels, have been described; the third, with S. H. Woodard, M. Am. Soc. C. E., as Resident Engineer, embraced all tunnels from the easterly end of the work near East Avenue in Long Island City to the meeting points under the river and also the permanent shafts in Long Island City. A few months after the execution of the principal contract, the work to be done was extended eastward 107.5 ft., across East Avenue. The extensions of the tunnels were built without cast-iron linings and with an interior cross-section of the same height as the tube tunnels, but somewhat narrower. The work was also extended westward from the First Avenue shafts to include the excavation of top headings in each tunnel for a distance of 100 ft. and an enlargement to full size for 50 ft. The borings having shown that soft earth existed below the grade of the tops of the tunnel under the passenger station building of the Long Island Railroad on the east side of Front Street, and that earth of varying character would be met in places beyond the station building under the railroad tracks in the passenger yard and the street car tracks in Borden Avenue, it had been decided, before proposals were invited, to extend the metal lining eastward to East Avenue, at the east end of the work embraced in the original contract, where the rising tunnel grades approached the surface of the ground so closely that their further extension would be in open cut. In places where the tunnels were wholly in rock, the weight of the cast-iron tunnel lining was reduced 43%; where the surface of the rock was below the top of the tunnel, but above the axis, the reduction of weight was somewhat less, about 25%; notwithstanding these savings, the cost of the tunnels was probably increased by the use of the cast-iron lining; on the other hand, when passing through bad ground, a section of tunnel could be made absolutely safe more quickly by erecting the lining as soon as a length of a few feet of tunnel was ready; under a crowded passenger yard, this feature had great value.

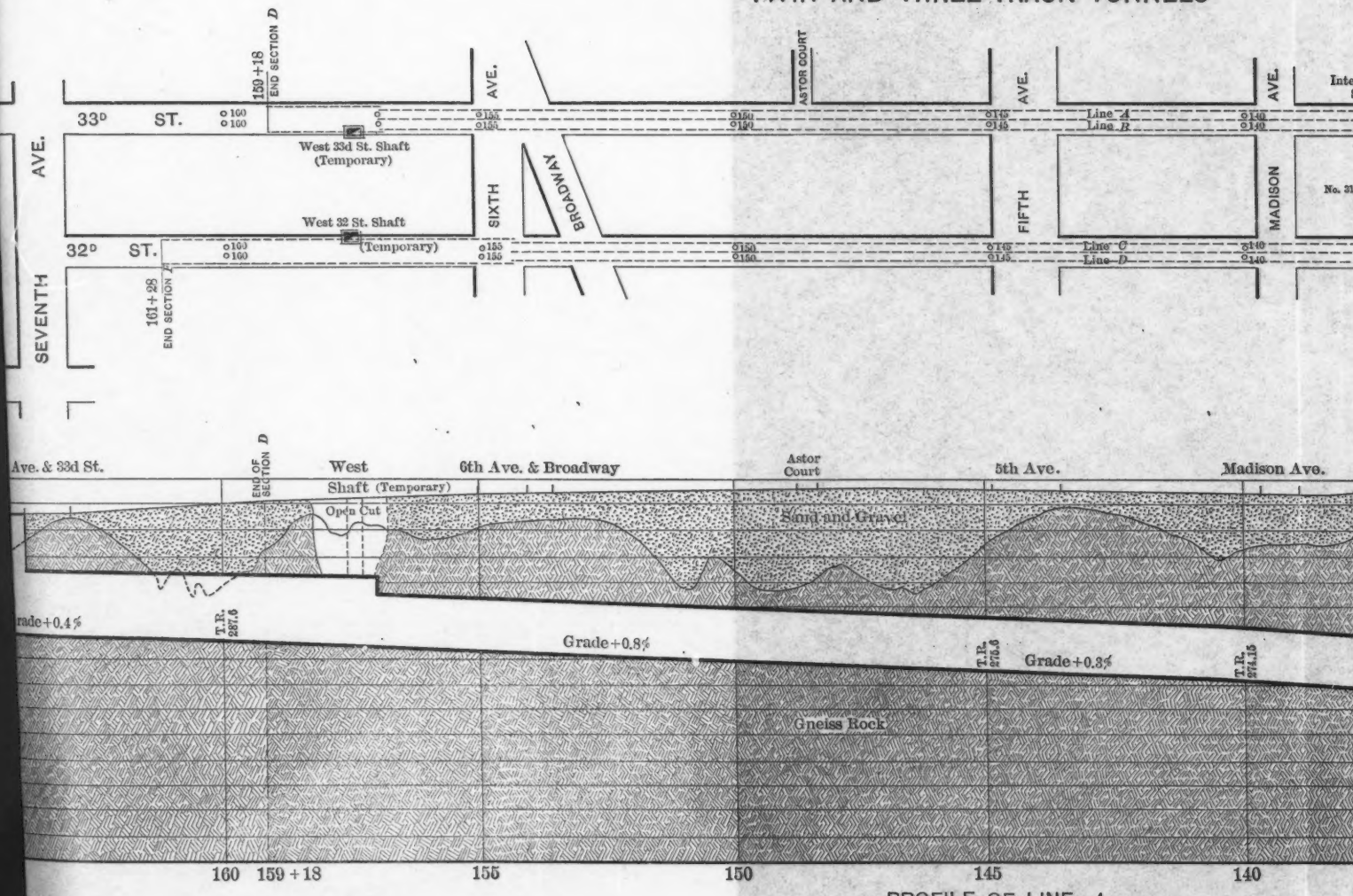
The execution of the work under this contract will be described fully by the Resident Engineers.

The plant assembled by the contractors is believed to be the most extensive ever placed on a single piece of work, and will be described in detail by their Managing Engineer, Henry Japp, M. Am. Soc. C. E.

For convenience in receiving materials to be used in construction, and to facilitate the disposal of excavated materials, one pier was leased on the east side of the Hudson River, two on the west side of

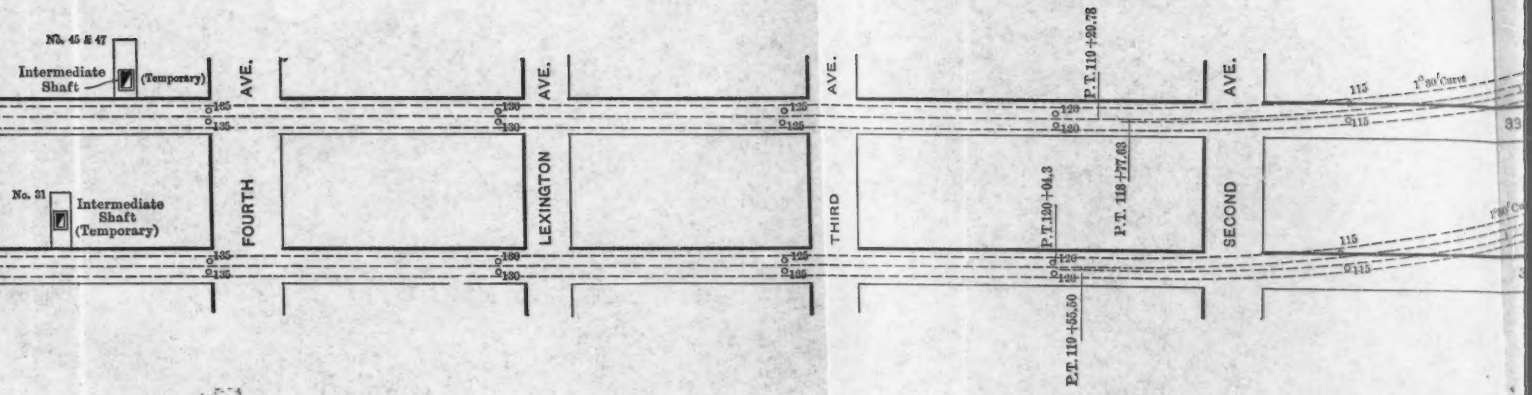

N
33D
ST.
AVE.
NINTH
TERMINAL
EIGHTH
STATION
SEVENTH
31ST.
ST.
159+18
END SECTION D
33D
ST.
West 33d St. Shaft
(Temporary)
West 32 St. Shaft
(Temporary)
32D
ST.
161+28
END SECTION C
SIXTH
BROADWAY
9th Ave.
8th Ave.
7th Ave. & 33d St.
Surface of 32d St. (Vacated)
END OF SECTION D
West
Shaft (Temporary)
6th Ave. & Broadway
Open Cut
Grade-0.4%
Grade+0.4%
Grade+0.8%
T.R. 287.6
350'
340
330
320
310
300
290
280
270
260
250
240
230
220
210
200'
180
175
170
165
160
159+18
155

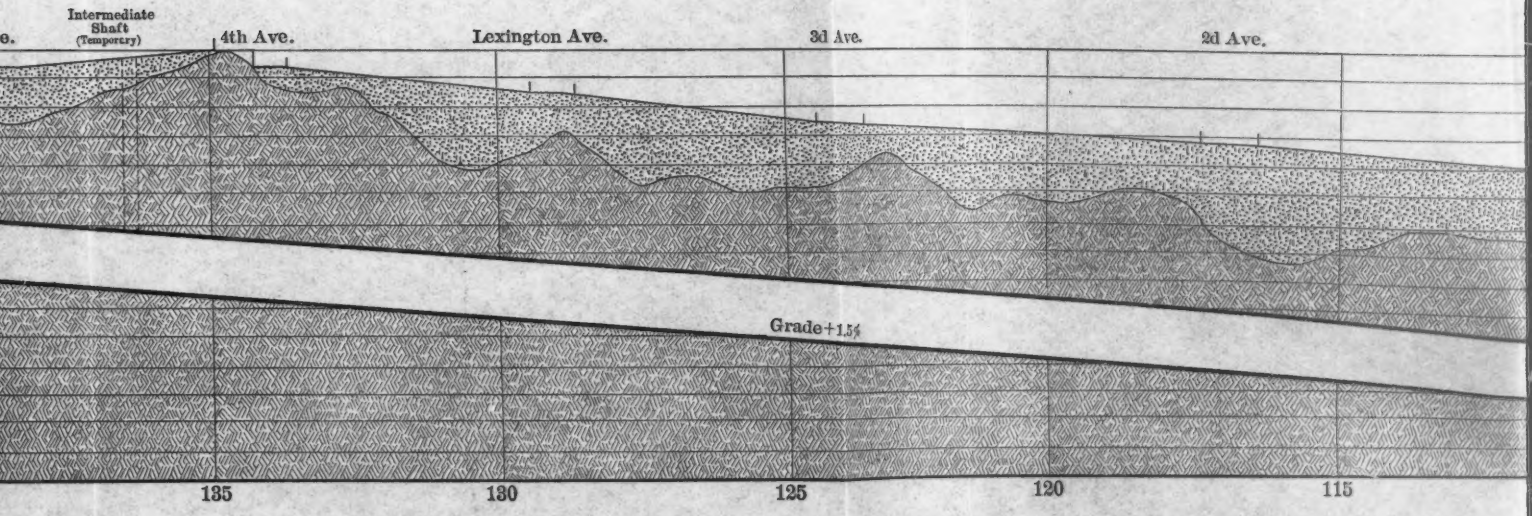
TWIN AND THREE-TRACK TUNNELS



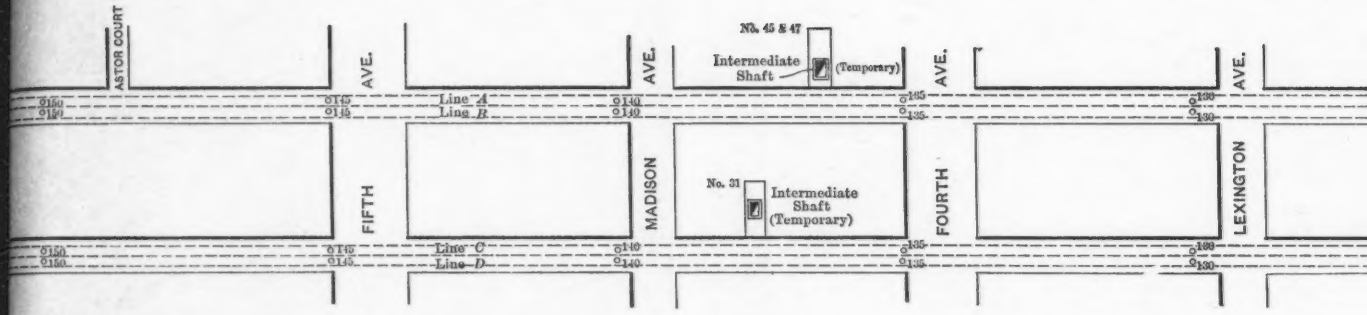
PROFILE OF LINE *A*

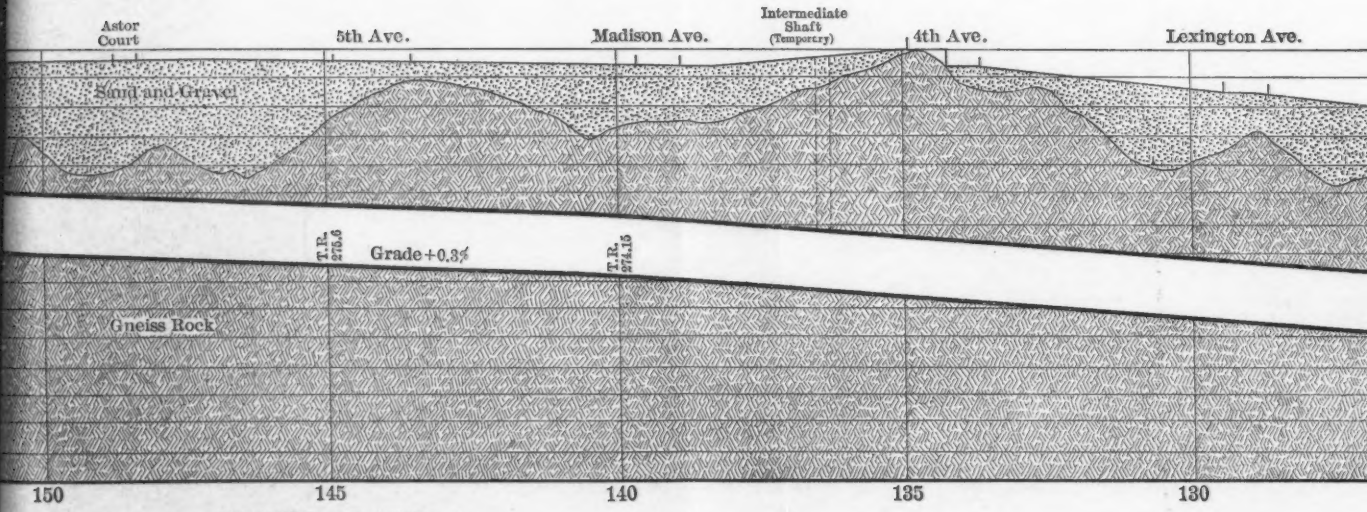
PLATE
PAPERS, AM
SEPTEMB
NOBL
EAST RIVER DIVISION: PEN



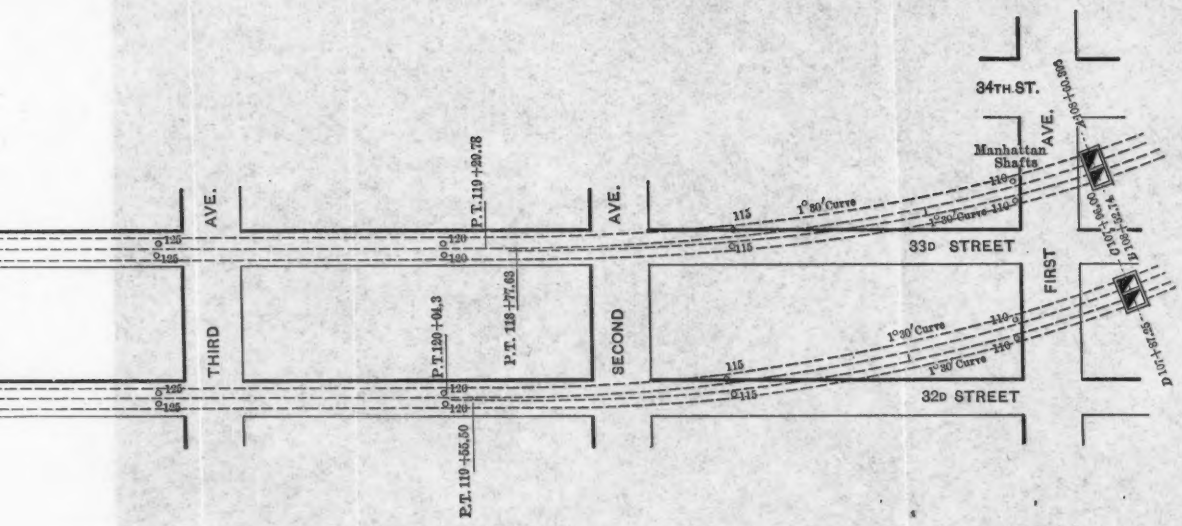


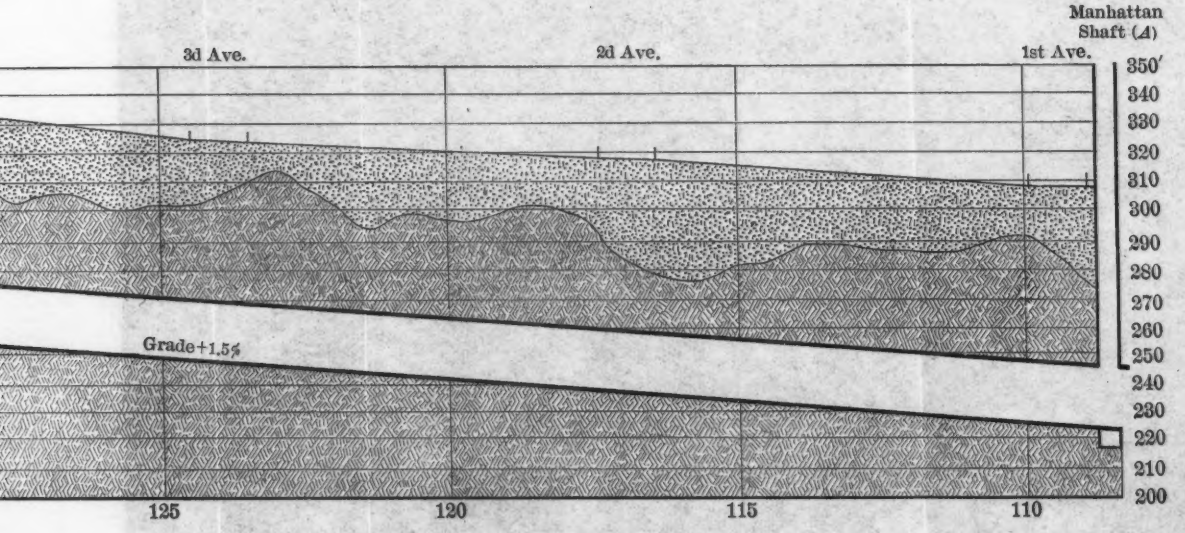
TWIN AND THREE-TRACK TUNNELS





PROFILE OF LINE *A*


34TH ST.
AVE.
Manhattan Shafts
33D STREET
32D STREET
THIRD
SECOND
FIRST
AVE.
AVE.
P.T. 119+89.78
P.T. 120+04.3
P.T. 119+77.63
P.T. 119+55.50
1°30' Curve
1°30' Curve
1°30' Curve
1°30' Curve
115
110
120
125


Manhattan Shaft (A)
3d Ave.
2d Ave.
1st Ave.
Grade+1.5%
350'
340
330
320
310
300
290
280
270
260
250
240
230
220
210
200
125
120
115
110

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over

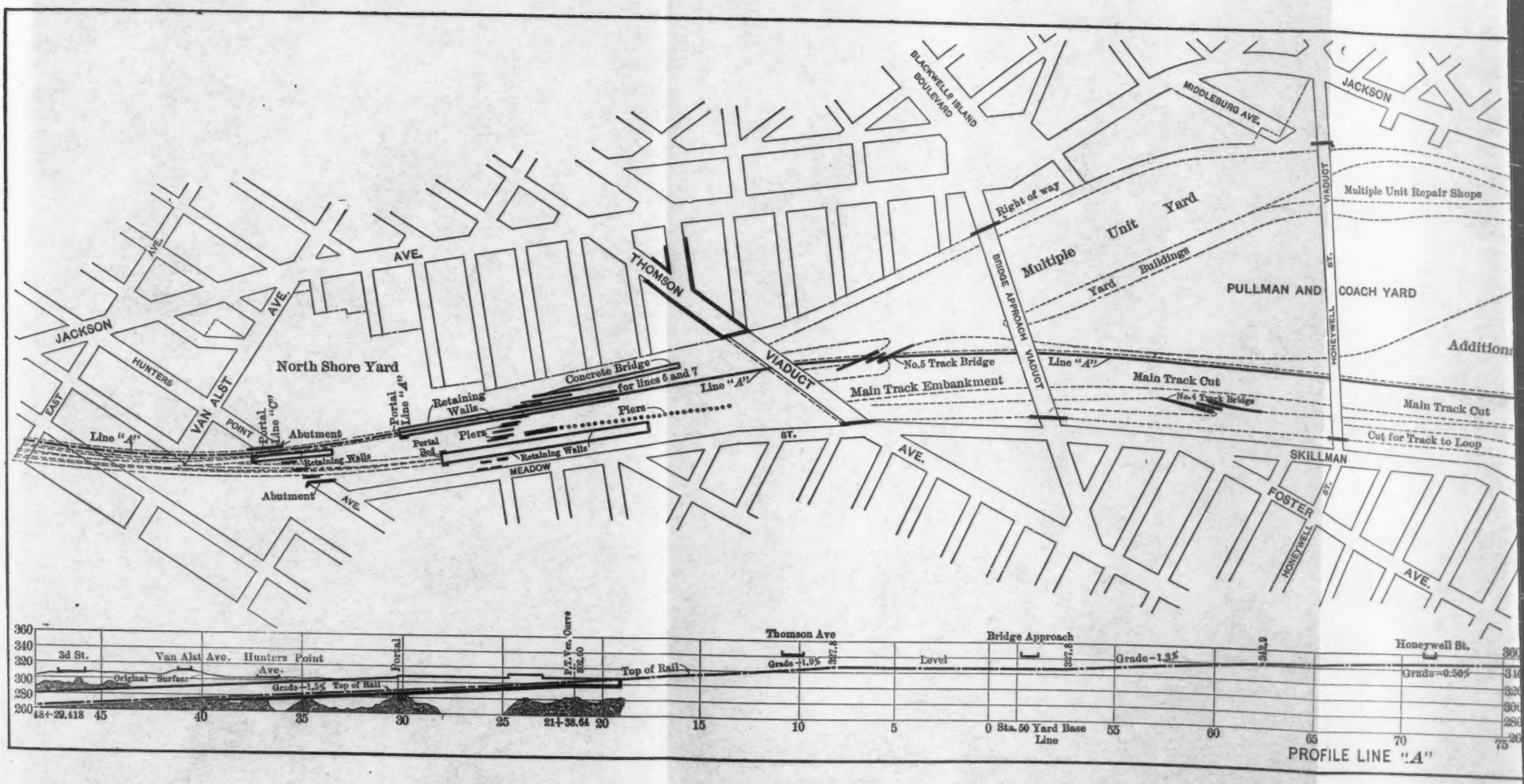
the East River and three on the east side. Excavated materials from the station, the cross-town tunnels, and the river tunnels, were placed on barges furnished by Mr. Henry Steers under several contracts embracing also the disposal of the materials. In the earlier part of the work, they were used as fill in the freight terminal of the Pennsylvania Railroad at Greenville on the west side of the Upper Bay; when the fill at this place was completed, the materials were sent to the tunnel company's yard on the Passaic, at Harrison, N. J., and a small part to the embankment in the Meadow Division. On account of the occasional closing of the Passaic by ice, this involved the possibility of, and to some extent resulted in, interruptions to the work of excavation. The contract for the cross-town tunnels carried an option in favor of the company to require the contractor for those tunnels to dispose of materials as a stated price, and in the latter part of 1907, when the excavation in these tunnels was being pushed rapidly, the railroad company, unwilling to incur the responsibility for delays during the winter, availed itself of this option. The disposal of materials was an important part of the work, and will be dealt with more fully by the Resident Engineers.

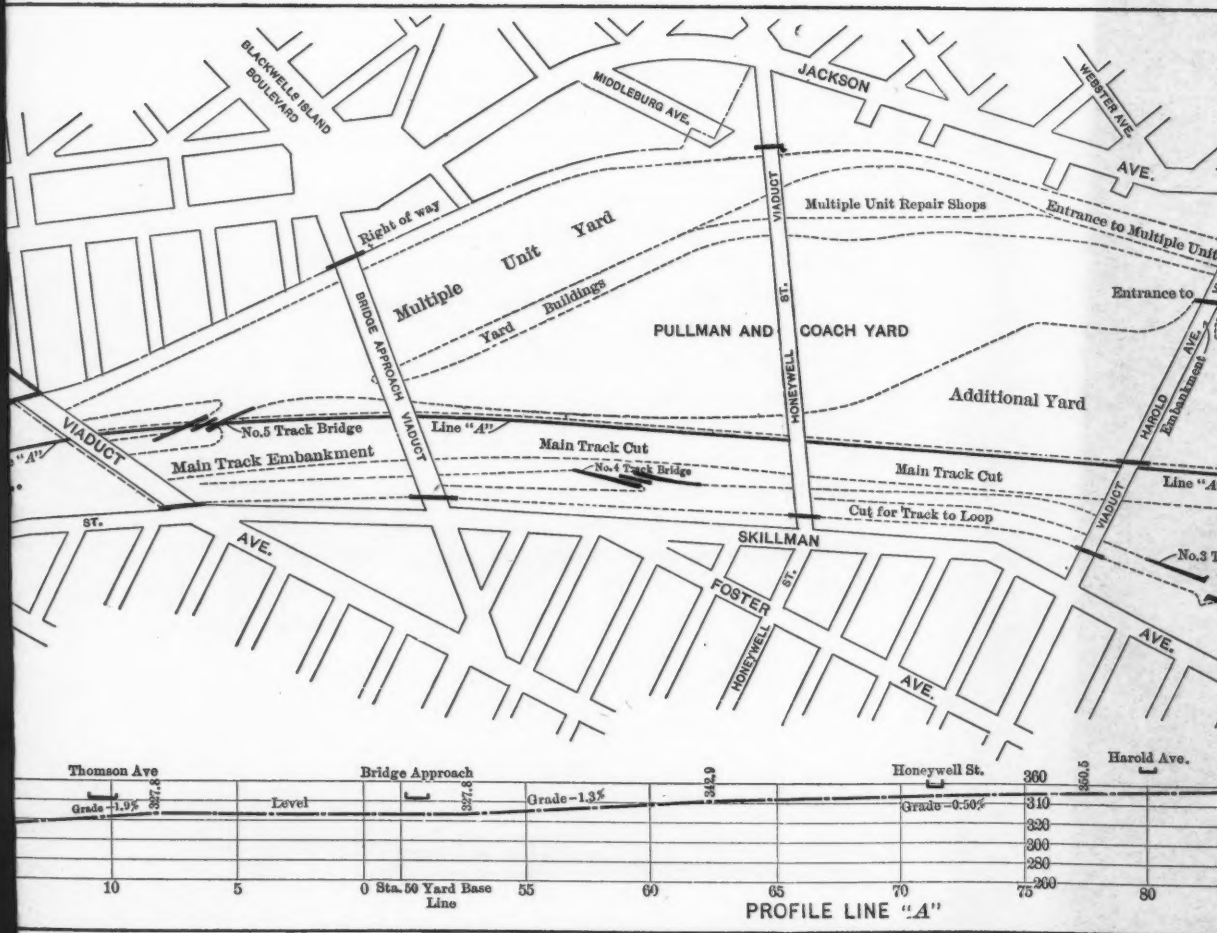
At the time the contract was made with S. Pearson and Son, Incorporated, it had not been determined whether mechanical ventilation would be provided for the tunnels, and therefore the contract with that firm did not include the final concrete lining at the shafts, above the inverts of the tunnels. After the adoption of plans for mechanical ventilation, in the latter part of 1908, the plans for lining the shafts with concrete, including flues for conducting air to the tunnels, and stairways for ingress and egress, were completed, and the work was placed under contract; it will be described in detail by F. M. Green, Assoc. M. Am. Soc. C. E.

At the east end of the work, under the Pearson contract, the rising grade of the tunnels brought them so near the surface of the ground that their extension eastward could be carried out more readily in open cut than by tunneling. The locations of the portals could be varied somewhat, and they were built on rock which was found in rather narrow ridges at convenient places. Tunnels *B* and *D* have a common portal; Tunnels *A* and *C* have separate ones, the portal for Tunnel *C* being located about 800 ft. west of the others as a result of its crossing over Tunnel *B*, as already explained. Eastward from the portals, the

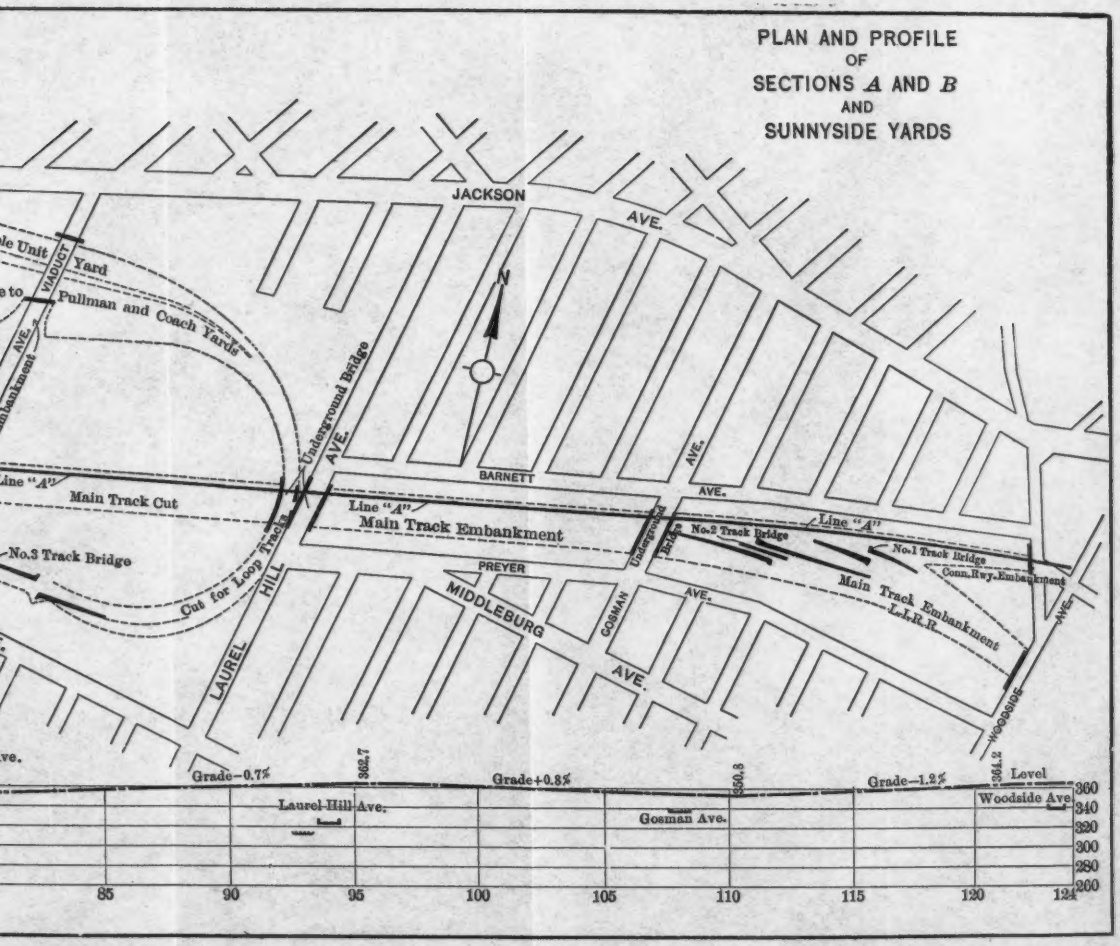
track system expands, in order to provide connections with the tracks of the Long Island Railroad to and from Long Island City, with the New York Connecting Railroad and New England lines, and with the storage and cleaning yard known as the Sunnyside Yard extending to the west side of Woodside Avenue, 2¾ miles east of the East River. (Plate LXVII.) The yard and approaches are designed to avoid grade crossings by opposing trains. The various general features of the yard and tunnel approaches, bridge crossings, and street closings, have been described in sufficient detail by General Raymond in the introductory paper.

For convenience in placing the work under contract, a line was drawn 10 ft. west of Thomson Avenue, dividing the work east of that embraced in the Pearson contract into two parts. The work west of the line was placed under the immediate direction of George C. Clarke, M. Am. Soc. C. E., as Resident Engineer, with Naughton Company and Arthur McMullen, Contractors; Mr. Louis H. Barker was Resident Engineer of the part east of the dividing line, with the Degnon Realty and Terminal Improvement Company as the principal contractors. The substructures of the several bridges in or across the yard were included in these contracts, but the superstructures were carried out by various bridge companies, and other minor features were executed by other contractors. More complete descriptions of the plans and of the execution of the work will be given by the Resident Engineers.


BLACKWELLS ISLAND BOULEVARD
MIDDLEBURG AVE.
JACKSON
Right of way
Multiple Unit Repair Shops
Multiple Unit Yard
Yard Buildings
VIADUCT
PULLMAN AND COACH YARD
HONEYWELL ST.
Addition
JACKSON
HUNTERS
EAST
VAN ALST AVE.
POINT
North Shore Yard
Portal Line "C"
Portal Line "A"
Abutment
Retaining Walls
Concrete Bridge for lines 6 and 7
Line "A"
Piers
Portal Bed
THOMSON AVE.
VIADUCT
BRIDGE APPROACH VIADUCT
No.5 Track Bridge
Main Track Embankment
Line "A"
Main Track Cut
No.4 Track Bridge
Main Track Cut
Cut for Track to Loop
MEADOW
Abutment
AVE.
ST.
SKILLMAN
FOSTER
HONEYWELL ST.
AVE.
3d St.
Van Alst Ave.
Hunters Point Ave.
Original Surface
Grade +1.5% Top of Rail
Portal
P.T. Vert. Curve 302.60
Top of Rail
Thomson Ave
Grade +1.9%
327.8
Level
Bridge Approach
327.8
Grade -1.3%
342.9
Honeywell St.
Grade -0.50%
360
340
320
300
280
260
48+29.418
45
40
35
30
25
21+38.64
20
15
10
5
0 Sta. 50 Yard Base Line
55
60
65
70
75
PROFILE LINE "A"



PROFILE LINE "A"


PLAN AND PROFILE
OF
SECTIONS A AND B
AND
SUNNYSIDE YARDS
JACKSON
AVE.
N
Yard
Pullman and Coach Yards
VIADUCT
Underground Bridge
AVE.
BARNETT
AVE.
Line "A"
Main Track Cut
Line "A"
Main Track Embankment
No.3 Track Bridge
Cut for Loop Tracks
LAUREL HILL
PREYER
MIDDLEBURG
AVE.
GOSMAN
Underground Bridge
No.2 Track Bridge
Line "A"
No.1 Track Bridge
Conn. Rwy. Embankment
Main Track Embankment L.I.R.R.
AVE.
WOODSIDE
AVE.
Grade-0.7%
362.7
Grade+0.8%
350.8
Grade-1.2%
361.2
Level
Laurel Hill Ave.
Gosman Ave.
Woodside Ave.
360
340
320
300
280
260
85
90
95
100
105
110
115
120
124

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INSTITUTED 1852

PAPERS AND DISCUSSIONS

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THE REINFORCED CONCRETE WHARF OF THE UNITED FRUIT COMPANY AT BOCAS DEL TORO, PANAMA.

By T. Howard Barnes, M. Am. Soc. C. E.

To be Presented November 3d, 1909.

Among the several tropical divisions of the United Fruit Company, whose home office is in Boston, Mass., that in the Republic of Panama is of rapidly growing importance. It is called the "Bocas Division," from Bocas del Toro, the Provincial Capital, where the division headquarters are located, and is about 140 miles northwest from Colon. It may be noted in passing that the name is illogical, for its translation is "Mouths of the Bull." There are indeed several mouths from Almirante Bay and Chiriqui Lagoon, the bodies of land-locked water which form the magnificent harbors pertinent to this Capital, whence naturally the plural, "Bocas," is suggested, but the ascription of them all to the bull is an unwarranted monopoly on his part.

The fruit product at this port consists wholly of bananas which are planted on the suitable portions of the alluvial formation along the rivers entering the Caribbean Sea between Almirante Bay and Punta Mona, in Costa Rica.

In the past the method of collecting the fruit, in addition to picking it up by train from the farm loading platforms and its subse-

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

quent train transportation, involved its transfer to lighters having a capacity of from 600 to 1 200 bunches and then towing by launches, *via* river and canal, about 12 miles to the ship anchorage in Almirante Bay. Here the fruit had to be lifted by stages to the side ports of the vessels for its stowage.

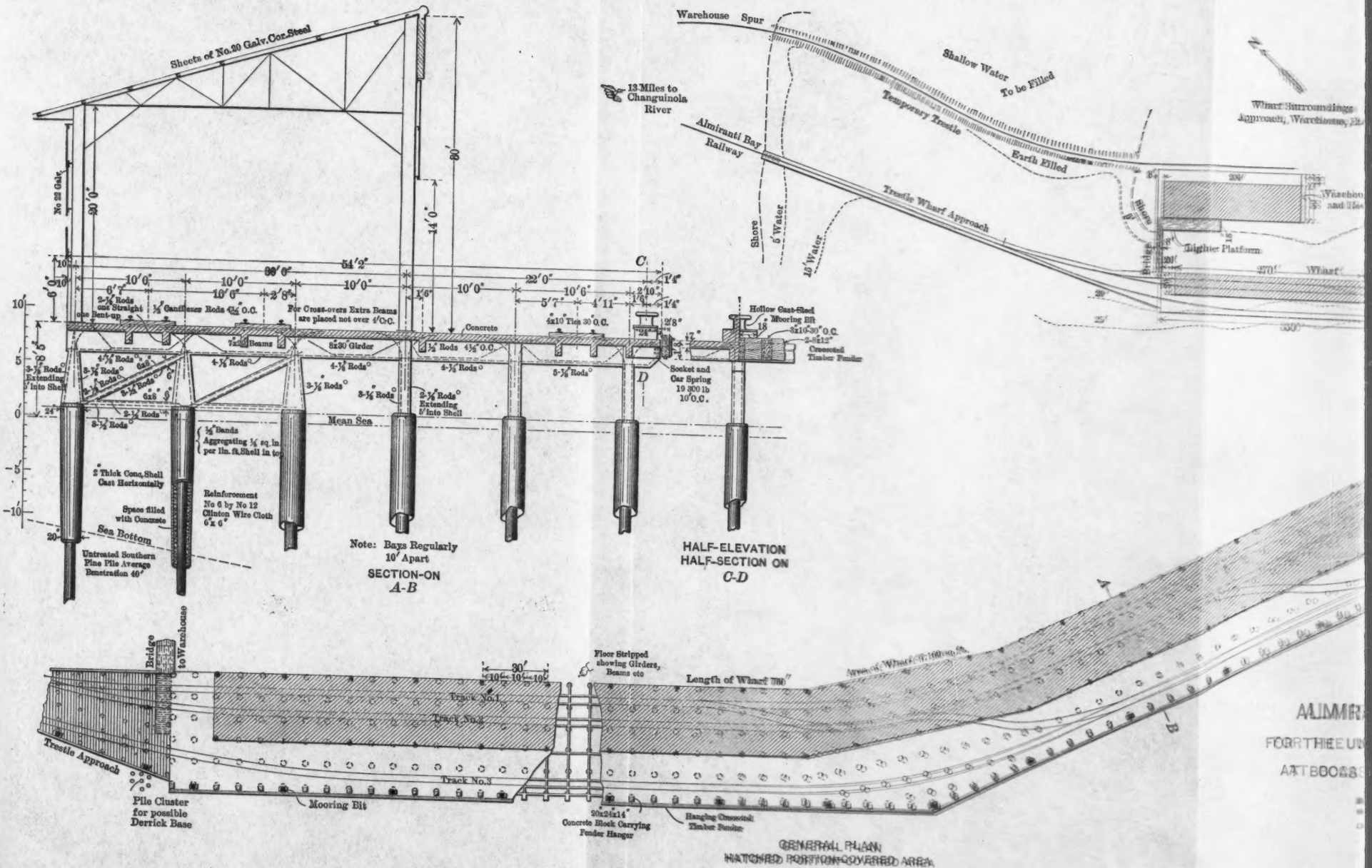
It was desired to avoid the expense and damage involved in this lighter trip and the handling of the fruit, besides which there was damage by scorching and salt water spray. To this end it was necessary to build 13 miles of railroad through difficult country, several steel bridges (one, with its approaches, being ¼ mile long), and suitable terminal works. The latter included a railroad yard, a two-berth wharf with a trestle approach 575 ft. long, a merchandise warehouse, water supply, etc.

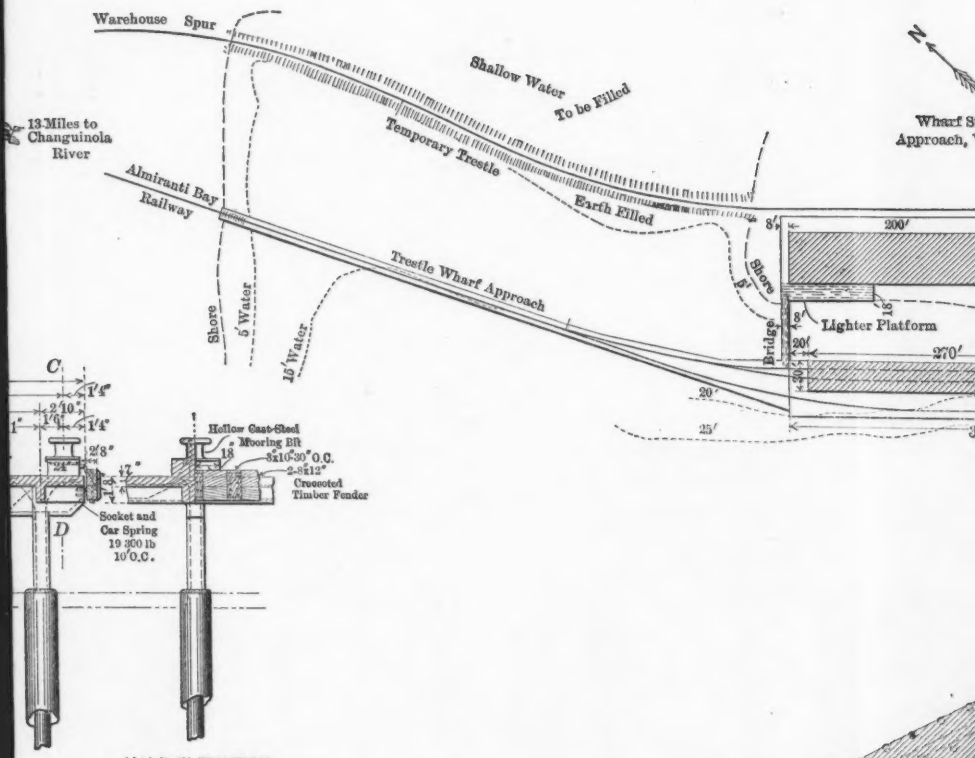
In September, 1906, the writer was engaged at Bocas del Toro in executing a contract for filling, sewering, and other sanitation work which the United Fruit Company had entered into with the Panama Government, and he was requested by the division manager to assume supervision of the construction of the wharf. Plans for a creosoted wooden structure were then under way at the home office, and it was expected to have it built under contract. At the same time the division manager further authorized the writer to prepare plans for a more permanent structure than the type proposed.

The *Teredo navalis* is the pest of wooden construction in those waters. So rapid is its work that ordinary native piles 10 in. in diameter are completely eaten through at the water line in a period as short as 8 months. Some varieties of wood last from 1 to 4 years, but these woods are scarce and of small size. The untreated southern pine quickly succumbs, it being noticeable that the teredo honeycombs the entire stick, including the resinous heart, leaving as the only outward marks the usual peppering of entrance holes perhaps $\frac{1}{30}$ in. in diameter.

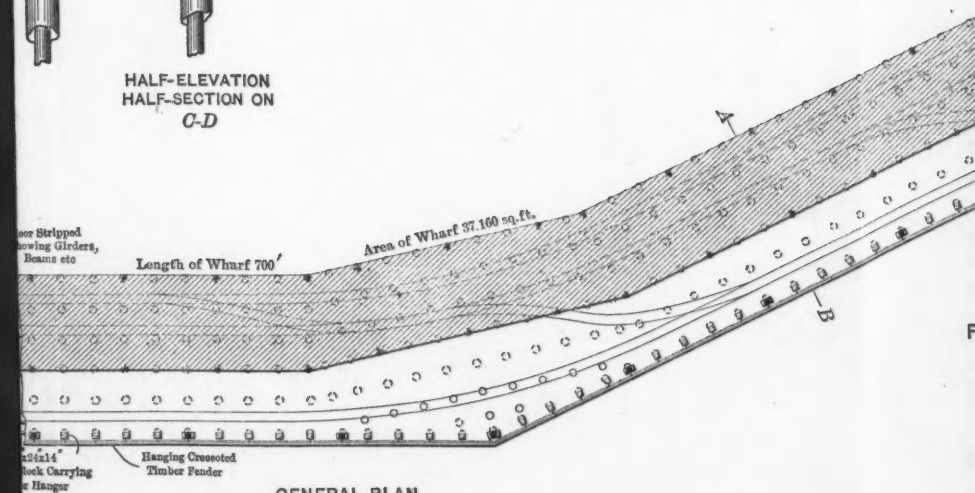
As is well known, the activity of the teredo is for the most part confined to the vicinity of the water line, or between high and low water. As observed by the writer, it does some damage at depths as great as 20 ft., but none at all below the sea bottom. Creosote is fatal to the teredo, consequently its work on a treated pile proceeds only as the outer shell of the pile loses its creosote by swash or otherwise.

Along the Caribbean Coast of Central America the tide is slight,


Sheets of No.20 Galv. Cor. Steel
No 22 Galv.
20'0"
14'0"
80'
Warehouse Spur
13 Miles to Changuinola River
Shallow Water To be Filled
Temporary Trestle
Earth Filled
Almirante Bay Railway
Trestle Wharf Approach
Shore
5' Water
15' Water
Wharf Surroundings Approach, Warehouses, Etc
Lighter Platform
Wharf
Bridge
Warehouse and Office
270'
300'
30'0"
54'2"
22'0"
10'0"
6'7"
2-¾" Rods one Straight one Bent-up
½" Cantilever Rods 4½" O.C.
For Cross-overs Extra Beams are placed not over 4' C-C.
7"x20" Beams
8x30 Girder
4x10 Ties 30" O.C.
Concrete
½" Rods 4½" O.C.
Hollow Coal-Shed
Mooring Bit
8x16"x30" O.C.
2-4x12" Creosoted Timber Fender
Socket and Car Spring 19 200 lb 10' O.C.
4-⅞" Rods
3-⅞" Rods Extending 5' into Shell
2-¾" Rods Extending 5' into Shell
Mean Sea
½" Bands Aggregating ¼ sq. in. per lin. ft. Shell in to
2" Thick Conc. Shell Cast Horizontally
Reinforcement No 6 by No 12 Clinton Wire Cloth 6"x 6"
Space filled with Concrete
Sea Bottom
Untreated Southern Pine Pile Average Penetration 40'
Note: Bays Regularly 10' Apart
SECTION-ON A-B
HALF-ELEVATION HALF-SECTION ON C-D
Bridge
to Warehouse
Trestle Approach
Track No.1
Track No.2
Track No.3
Floor Stripped showing Girders, Beams etc
Length of Wharf 770'
Area of Wharf (2.160 sq. ft.)
Pile Cluster for possible Derrick Base
Mooring Bit
20x24x14" Concrete Block Carrying Fender Hanger
Hanging Creosote Timber Fender
GENERAL PLAN
HATCHED PORTION COVERED AREA

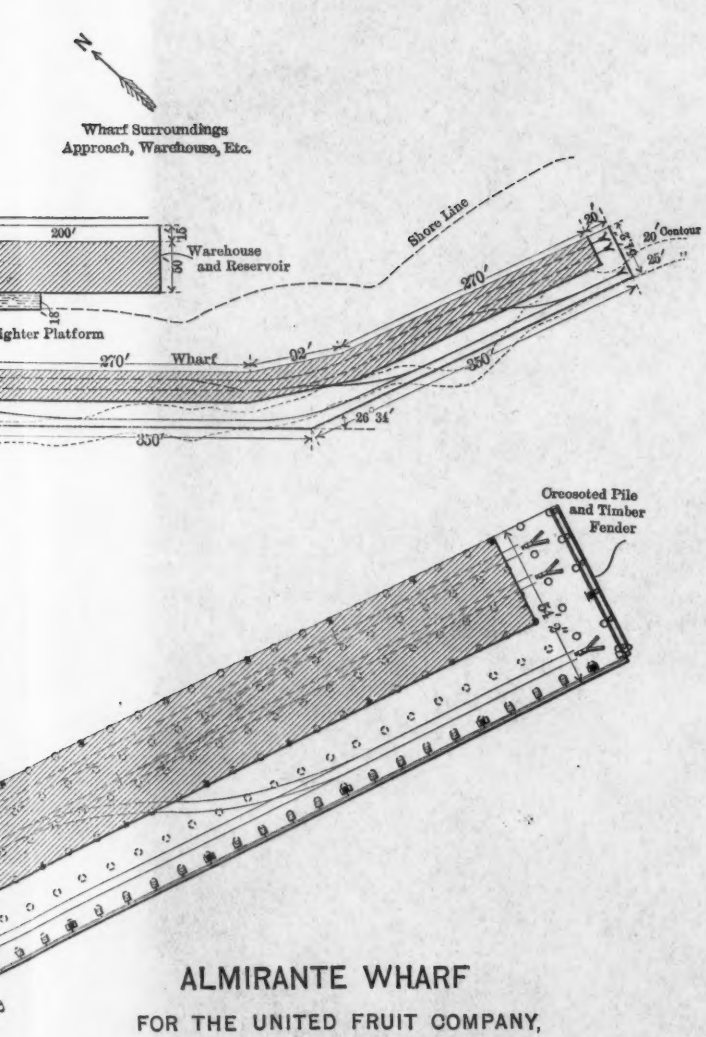


HALF-ELEVATION
HALF-SECTION ON
C-D



GENERAL PLAN
HATCHED PORTION=COVERED AREA

PLATE LXVIII.
PAPERS, AM. SOC. C. E.
SEPTEMBER, 1909.
BARNES ON
REINFORCED CONCRETE WHARF AT BOCAS DEL TORO.



ALMIRANTE WHARF

FOR THE UNITED FRUIT COMPANY,

AT BOCAS DEL TORO, PANAMA.

■=Mooring Bits
●=Columns in Shed
○=Shell and Pile

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perhaps nowhere having a fluctuation of more than 15 in.; in Almirante Bay it averages 9 in.

There may perhaps be taken, as representative of the extremes in durability of creosoted piles on this coast, the experience at Port Limon, Costa Rica, which is little more than a roadstead, and Puerto Barrios, Guatemala, where there is a land-locked harbor. In the former port there is an almost constant ground swell which produces a violent swash. Some unprotected creosoted pine piles driven for fender purposes along the National Wharf were so badly eaten at the end of 5 years as to require replacing. The regular bearing piles here are copper-sheathed over felting, and have suffered not at all from the teredo, but have been badly eaten at points where the coppering did not extend into the sea bottom. In Puerto Barrios Harbor, where there is only a wind chop to produce a swash, creosoted piles have lasted 15 years before requiring renewal.

While it is true that in Almirante Bay no ground swell enters the part used as an anchorage, and much less the bight where the new wharf is built, nevertheless, a tenure of life of 15 years is short in view of the interruption to business caused by reconstruction; and, in respect to a copper protection, its expense would have added from 15 to 20% to the cost of creosoted construction, and coppering is not without constant expense for repairs. The writer immediately took up the question of a reinforced concrete type of construction, securing Joseph R. Worcester, M. Am. Soc. C. E., as Advisory Engineer. Various types of reinforced concrete piles were considered, but always with a hesitancy to adopt their use in view of the extreme length demanded, viz., 70 ft.

Finally, the writer made studies for using an untreated southern pine bearing pile, to be protected with pipes or shells of reinforced concrete. The shells were planned to be in lengths of about 4 ft., and having a diameter 6 in. greater than the outside of the pile. It was expected to use a diver in placing these pipes. The annular space was to be filled with lean concrete or even sand, embodying the steel reinforcement for the columns, at the pile heads, in richer concrete. The pile, thus enveloped and protected, was to constitute the base for the column and the imposed superstructure. Afterward, the plans were modified, and a single pipe or shell of tapering dimensions was adopted, as will be described later; otherwise, they were executed as outlined.

The estimated cost of a structure of this type was about the same as for a plain creosoted wooden pile and timber construction. The notable saving in favor of the reinforced concrete type lay in the smaller cost of the untreated, as compared with the creosoted pile.

When the new plans and estimates were presented to the home representative assigned to this division of the company, the decision to adopt that type was quickly made, and the manager was commissioned to execute the construction on company account.

Location of the Wharf.—The situation, shape, and immediate surroundings of the wharf are shown on Plate LXVIII. The bottom near the shore is steep, so that the depth of the water is soon 50 ft. or more, a depth impracticable in which to build, and a condition which determined the location as shown, parallel with the contours, and affording berths on only one side of the wharf. It is to be observed that the depth of the water varies from about 10 ft. along the rear to about 22 ft. along the front of the wharf, affording a depth of about 27 ft. a ship's half breadth seaward therefrom.

Foundation.—The nature of the bottom was unknown, excepting by hand soundings. Accordingly, actual bearing tests were made on four 50-ft. piles driven in 9 ft. of water in permanent locations. A No. 3 Vulcan steam hammer was used on a floating driver. The total weight of the hammer and frame was 3 800 lb., the hammer proper weighing 1 800 lb. and having a throw of 30 in. About 130 blows per pile were used, the penetration of the last blows averaging $\frac{1}{2}$ in. The 3 800 lb. of dead weight sank the piles about 12 ft., leaving about 29 ft. of driving.

The sea bottom shows live coral on the surface, and all the indications are that coral in defunct forms extends as far as the piles penetrated. In line with this judgment it may be stated that existing shoals nearby (which have been excavated to a depth of 15 ft.) showed fingers of coral mixed with heads of the brain type.

The application of 11 tons per pile produced no settlement. A load of 16 tons produced a settlement of $\frac{1}{2}$ in., which did not increase with time, while 21 tons applied to each pile produced no further immediate settlement, but did produce one varying from $\frac{3}{4}$ in. to 3 in. in 4 days.

Load for Which to Provide.—The railroad locomotives constituted the chief concentrated load to be provided for. Owing to the easy grades

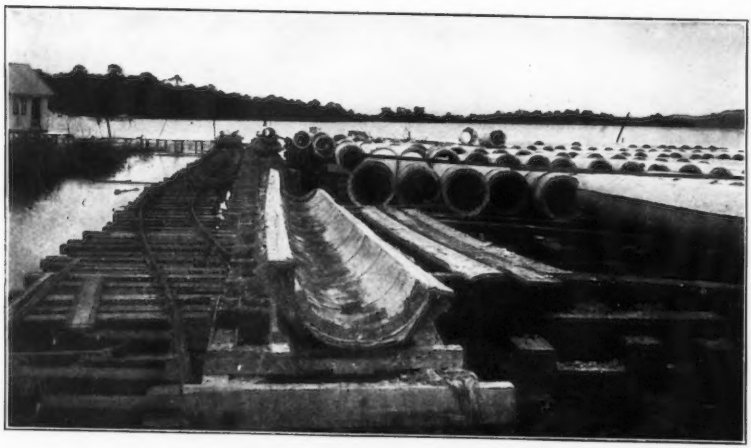

FIG. 1.—ALMIRANTE WHARF: SHELL RUNS, RECEIVING END.

FIG. 2.—ALMIRANTE WHARF: SHELL RUNS, DELIVERY END.

prevailing on the railroad, no locomotives exceeding 24 tons on the drivers of 11 ft. wheel base are required. With a spacing of 10 ft., a maximum concentrated live load of 6 tons per pile would be imposed. The dead load of concrete amounts to 9 tons per pile. A distributed live load much in excess of 6 tons per pile might occur at rare intervals, and the slab was calculated to sustain 250 lb. per sq. ft. During construction the wharf underwent an actual test closely approaching this, owing to the storage of some cement thereon. The capacity of the girders and beams to enlist through their rigidity the support of adjacent piles otherwise taking little live load may be depended on to increase the factor of safety. These considerations led to the neglect of a load of 3 tons imposed on each pile where a shed column should bear. The spacing adopted was 10 ft., and this was maintained as far as the shape of the wharf would permit.

Uses of the Wharf.—Plate LXVIII shows the general construction of the wharf, the spacing of the piles, the arrangement of girders and beams, the location of the tracks, shed, etc. The design of the approach and tracks, shed, etc., is calculated to meet the demands for the prompt handling of fruit. Tracks 1 and 2 are for fruit, being under cover and furnished with ample cross-connections, whereby it is possible to keep a train of fruit constantly on one of the tracks at the berth in use, while cargo from the other berth may be taken off. At the present time, as no loading device has been adopted, and the passing of fruit by hand through the side ports is in practice, the outboard track (Track 3) is utilized for fruit loading, but machines will soon be used for carrying the fruit from Track 2 to the various decks of the vessel, and Track 3 will remain, as originally planned, to take incoming cargo for transfer to the warehouse.

General Description of the Wharf.—The framing system comprises bays of six columns capped with 8 by 30-in. girders, with longitudinal stiffening of 7 by 20-in. beams, one under each rail, as well as many others for cross-overs, unloading-machine bearings, and other stiffening. The dimensions of the wharf are shown on Plate LXVIII, the length of each berth being 350 ft. on the front.

The three front rows of columns are 12 by 12 in. in section, while the rear rows are tapered from 8 by 24 in. at the top of the piles to 8 by 12 in. at the girders. The latter columns were thus shaped to grip the heads of the piles in a direction transverse to the wharf, and

constitute bracing piles by means of the struts and braces. A cross-section of the wharf and shed is also shown on Plate LXVIII, and this gives a better idea of the bracing than can be obtained by written description.

The columns are 4 ft. 10 in. high, making the wharf floor 8 ft. 6 in. above mean water.

The floor slab is 7 in. thick. The thickness of the girders is increased to 20 in. at the front, and they are bracketed up from the front row of piles. In this thicker mass is moulded a cast-steel cup, designed to receive a standard car spring of 19 000 lb. resistance under a compression of 2 in.

Fender.—The front of the wharf is protected against friction and shock by an 8 by 24-in. bolted-up wooden fender. The fender is suspended from anchor-bolts, set in concrete blocks, by a link and hook-bolt passing through the fender, and rests against the spring. The contact is protected by a 12 by 12 by ½-in. plate set flush with the inner face of the fender.

Actual use demonstrates the efficiency of the fender, and that the shock, to be avoided as possibly disastrous, is to the ship and not to the wharf, for the floor acts as an immense horizontal beam which unites the resistance of all the piles, and, being supplemented by the existing dead weight, opposes the momentum of the ship. It is exceptional, in fact, for an impact to cause more than the closing of the spring in one or two sockets.

Unit Stresses.—The extreme fiber stress allowed in the concrete was 600 lb. per sq. in. in compression, and no account of tensile resistance was taken. An allowance of 16 000 lb. per sq. in. in tension was made for the steel reinforcement.

Reinforcement.—Round bars of mild steel of two sizes were used for reinforcement throughout: ⅞-in. for columns, girders, and beams, and ½-in. for floor slabs. Of the bars in girders and beams, one-half were cantilevered over the point of support. As before stated, the column reinforcement was planted in the annular space between the pile and the shell, and was sufficiently long to reach into the floor slab. No roundabout reinforcement of the column was used, because of its shortness; nor was any special reinforcement used in the girders and beams to meet shear strains, as the bent-up bars amply provided for this resistance.

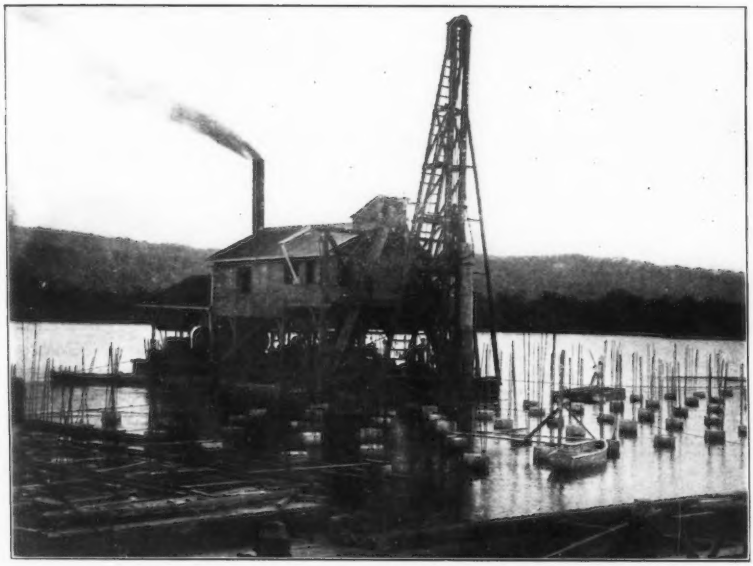

FIG. 1.—SHELL PLACING, ALMIRANTE WHARF.

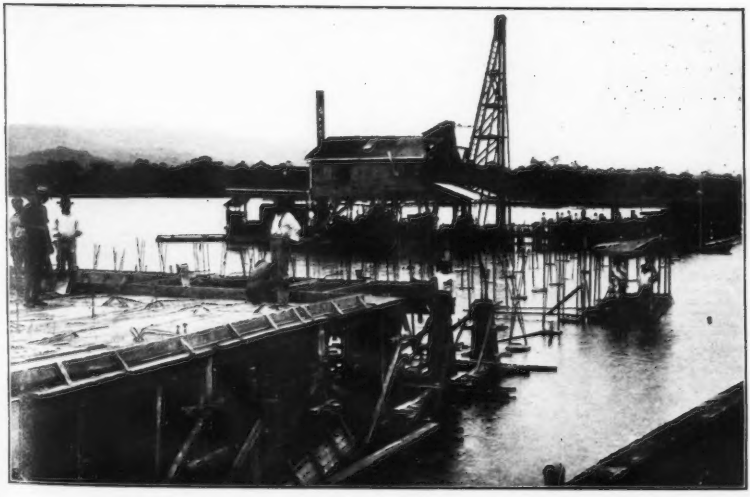

FIG. 2.—SHELL FILLING, FORM ERECTION, AND PLACING OF FLOOR CONCRETE, ALMIRANTE WHARF.

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Considerable extra reinforcement was introduced into the 8 or 10-ft. strip of floor along the wharf front, in order to meet an unknown strain which a listing vessel might impose on the slab midway between the girders.

Anchor-bolts for the 4 by 10-in. by 5-ft. railroad ties were built into the slab, one for each tie, in staggered position.

The reinforcement of the shells was, for the most part, the electrically-welded fabric made by the Clinton Company. The mesh best adapted, and the one principally used, was 6 by 6-in. with No. 6 longitudinal and No. 8 roundabout wires. Some few home-made spirally-wound cages were made for shell reinforcement, but they proved to be expensive and were difficult to use.

Materials of Construction.

Piles.—Untreated piles of southern pine were used throughout the wharf proper. In the trestle approach, 575 ft. long, creosoted southern pine piles were used. The former were especially selected for their uniformity and the straightness of the butt.

Stone.—The stone used was a dense trap having a slight cleavage, but generally breaking crosswise thereof, and constituting a good ingredient. It was quarried on an island about 6 miles from the work, there loaded into small lighters, and towed to the site.

Sand.—The sand was brought on lighters from the Changuinola River. It was the best to be had, and, while very good, was not as sharp as one would select had there been other kinds from which to choose. For this reason, screenings consisting of fine stone and dust from the crusher were used freely, and much extra expense was incurred in re-crushing the larger stone for the production of this grade. In fact, no effort was spared to secure concrete of a superior quality.

Cement.—Alsen and Vulcanite brands of cement were used with entire satisfaction. Field tests only, from actual mixtures, were made. The total quantity of cement used was 3 450 bbl.

Concrete.—An average mixture of 1:2:4 of cement, sand, and stone was used, the sand usually containing some crusher dust and fine stone, which would make the mixture really $1:1\frac{1}{2}:4\frac{1}{2}$. The idea of producing a dense concrete was kept foremost, especially in making the shells, on which any destructive effect of sea water would be mani-

fest. The ingredients were varied to suit the requirements for particular portions of the structure, being made especially fine and rich in the columns and in the bottoms of girders, beams, and slabs.

From time to time, 4-in. sample cubes were taken from the concrete as mixed. Four of these cubes were tested by the Worcester, Mass., Polytechnic Institute, with the co-operation of Chester S. Allen, Jun. Am. Soc. C. E., who was Assistant Engineer on the construction. The following are the results, the samples being 15 months old when tested: 2 367, 4 911, 3 355, and 5 367 lb. per sq. in., an average of 4 000 lb. per sq. in.

Details of Construction.

Plant.—The plant available in the division consisted of a No. 2 portable Climax crusher, an engine, an elevator, and bins. These were utilized, although they were not what would have been selected as first choice. The working base was an island shoal, the soil of which was very yielding, thus causing much annoyance and expense. In spite of 2 ft. of filling which had been pumped on the area, settlement took place, so that high tides covered much of the ground. The crusher engine was removed from its position on the boiler and set on a pile and concrete base. A rope drive from the engine actuated the No. 2 Ransome concrete mixer and hoist.

A 28 by 75-ft. hull was used for the floating pile-driver. The latter was equipped with leads 56 ft. high and the Vulcan No. 3 hammer previously described. A land driver was also brought into requisition for some driving for the warehouse and other structures on shore.

Shell Forms.—The 4-ft. shells, or rather pipes, which were at first planned to be used for pile protection, were never made, but, as stated previously, single shells long enough to reach from above high water to below the sea bottom were substituted, and these were reinforced so as to stand handling and placing by the pile-driver.

Soundings were taken at each pile location, and the shells were made of sufficient length to permit their sealing into the sea bottom and to reach above ordinary high water. Furthermore, the shells were made and stacked in the order in which they were to be used, thus greatly facilitating their placing.

The attempts to utilize adjustable forms and home-wound wire

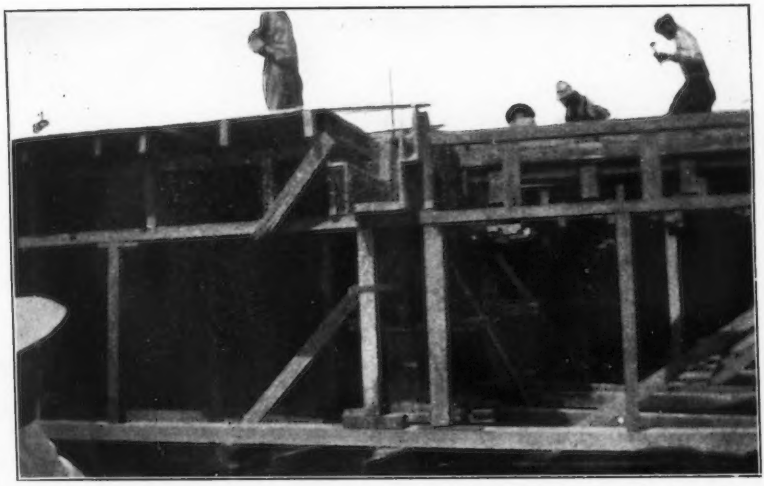

FIG. 1.—FRONT VIEW, ALMIRANTE WHARF, SHOWING SHORING, ETC.

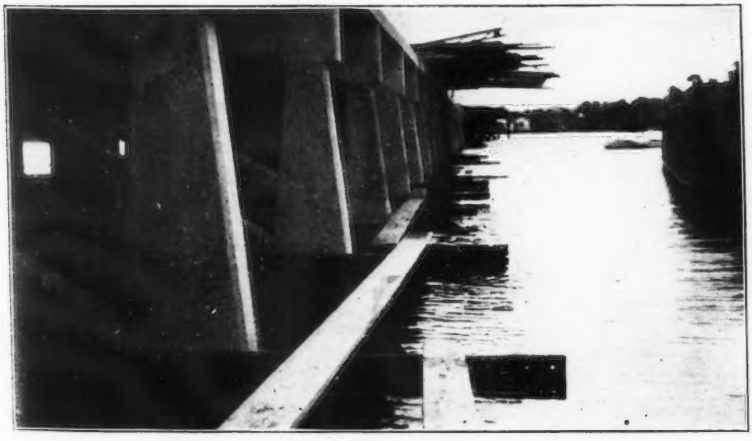

FIG. 2.—REAR VIEW, ALMIRANTE WHARF, SHOWING FINISH.

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reinforcement need not be described; but it should be noted that such forms must be very firm and stiff, and capable of preserving the desired thickness of shell, qualities which adjustable forms do not possess; and that it is too difficult to adjust any closed wire cage over the core. The forms adopted leave improvements to be made. They consisted of five parts: two half cylinders of No. 16 gauge galvanized iron, a spider or spacer of wood, a wooden cradle lined with sheet metal, and a wooden arch faced with sheet metal.

The half cylinders—really not cylinders but frustums of a cone—when fixed over the spider formed the core of the shell. They tapered from 20 in. to 16 in. in diameter, whatever the actual length might be. The cradle rested horizontally on a truck, and, together with its companion arch, formed an outer mould having a diameter 4 in. greater than the corresponding core, thus giving a thickness of shell of 2 in.

The exact length of shell desired was secured by stops, in the form of collars, made of short sections of wood attached to sheet metal. They served the further purpose of keeping the core concentric with the outer form. The forms were made up in lengths of 16, 18, 20, 24, 28, 30, and 32 ft.; there were several of each length, excepting of the longest ones.

Shell Making.—The stone ingredient of the concrete was necessarily limited to a size not exceeding ¾ in. in diameter. By careful grading, a mixture yielding 13 cu. ft. of concrete to the barrel of cement was obtained. The order of making was first to place the proper thickness of concrete along the bottom of the cradle, and therein embed the reinforcement. The latter opened longitudinally, and was wired together when the next step was taken, that of setting the core in place. After closing the wire about the core and clamping down the latter, the concrete was filled in to the "springing line" of what constituted the arch of the shell as it lay in a horizontal position. The arch form was then clamped on, and the remaining concrete was placed, openings along the top of the arch form having been left for this purpose. The concrete was compacted by churning it into place with light flat iron bars and by tapping the outer moulds. This method was effective in expelling the entrained air, and made a very solid concrete. Where in contact with the forms, the concrete was left very smooth, and the only additional finish was a wash of grout.

The open spaces at the top were necessarily left to be finished by hand. The forms were slightly oiled to assist in the removal of the shell.

The freshly moulded shells were carefully rolled along the track a minimum distance, and allowed to stand over night.

Shell Curing.—While the concreting was going on, two other gangs were at work, in addition to those mixing and placing the concrete. One small gang was preparing the cores and placing the stops, and a larger gang, equal to that required at the moulding, was busy unloading and stacking the shells. The concrete set very rapidly, so that all shells excepting those 25 ft. or more in length could be unloaded on the storage runs the morning after they were made. The spider was drawn first, and then the core forms, the upper one of the latter requiring ordinarily tedious cutting away of the fine rich concrete which had oozed into the lap of the upper over the lower portion of this form. Then the top or arch form was removed, and the cradle was rocked on its side, thus throwing the shell out on the run. Here it was inspected and patched, the latter being required in but few instances, and was kept moist for several days (Providence greatly assisting). The runs consisted of two lines of supports along the shells at their quarter points. A slight inclination toward the water facilitated the movement of the shells on the runs.

As stated, most of the shells were less than 24 hours old when they were unloaded; the longer ones were a day older, yet so rapidly does concrete set in an air temperature ranging from 75° to 90° in the full 24 hours, that the total losses from all causes amounted to only 2 per cent. Each shell was marked with a letter and number indicating its position.

The description of this process of producing the shells sounds simple in its recital, and so indeed it is, but the forms were heavy and the preparation was somewhat tedious; the men needed constant watching and spurring on to their task, and the result was that, with delays occasioned by using the moulding force to hurry up the preparation of the forms, or to unload some of the shells, it was possible to produce regularly not more than six shells per day. Altogether, the production of the shells was the hardest portion of the entire work.

Shell Placing.—The placing of the shells proved to be very easy. A special box punt, decked over, was used for the conveyance of the shells. This was tied broadside at the foot of the run, and the shell

FIG. 1.—FRONT OF ALMIRANTE WHARF BEFORE FIXING FENDER.

FIG. 2.—FRONT OF ALMIRANTE WHARF SHOWING FENDER HANGING.

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was rolled thereon over short skids. The punt was towed by rowboat to the pile-driver, and the shell was lifted by the fall with a rope sling into vertical position and placed about the pile just driven. The shell was allowed to drop with a slight momentum in order to bed the toe, and any further settling needed was secured by resting the hammer and frame on the shell top, or by slight drops of the hammer, with a cushion of rope and plank interposed. In some cases the shell was not long enough, indicating a settlement of the coral about the pile. In these cases, a sheet-metal casing was clamped about the head of the shell, and, usually with some difficulty, the head was extended to the proper grade. The time consumed in placing the shell was about six-tenths of that used in driving the pile.

Shell Filling.—After wedging the shell firmly in a concentric position about the pile head, a sufficient quantity of fine, rich concrete was deposited quietly within to seal about 2 ft. in depth at the foot of the shell. After two days or more it was generally possible to pump out the enclosed water, and then, under dry conditions, the lean concrete could be placed. This mixture was carried to within 5 ft. of the top, where the column reinforcement was embedded. The concrete for this portion was made of the rich mixture. Then the pile with its concrete envelope was jacked and clamped into position ready for the forms of the superstructure.

Concrete Forms.—The forms were made of $\frac{7}{8}$ by 4-in. matched lumber, with battens disposed for clamping by yokes and wedges. The shoring of the floor hangers (joists) and beam bottoms was carried on a system of railroad iron resting on the shell tops, with beams laid crosswise on the rails. The entire erection, with its yoking and shoring, demanded conscientious attention, even old hands not always realizing the necessity for inspecting carefully each and every point of bearing, while, as for novices, one needed the search-light eyes of the crab to supplement their delinquencies.

The raising of a system of centering as involved as that required to shape short columns and 6 by 8-in. braces is a tedious operation at best, but, when conducted over the water, fortitude and patience are prime essentials. The forms were used repeatedly, and in most respects served their purpose admirably. The forms provided were sufficient to center 110 lin. ft. of the wharf, entailing their use six times or more. They were given a light coat of oil at each erection after the first.

Placing the Concrete.—The mixture in the shells and other portions was placed quite wet, which was necessary in order to fill properly the spaces in areas of small cross-section like those dealt with. The method of churning and tapping proved most effective in the columns, braces, girders, and beams, in which portions the depth of the concrete assisted in its consolidation.

Constant vigilance was required to secure proper results. The Jamaican negro seems to think that, when he has once been through any operation properly, "that settles it, it's done forevermore." A smooth but not a polished surface was given the floor slab; this was done by local masons, but they too required like supervision in order to secure proper results.

The columns were generally moulded 24 hours in advance of the girders and beams; and the period between the latter and the floor slab was seldom less. The placing of the floor slab concurrently with the girders and beams produced a shrinkage crack along the plane of the bottom of the slab. An interval as short as 6 hours was at times allowed, but, as a rule, a longer period was maintained, although the defect mentioned did not appear after 6 hours' setting.

The work of the day on beams and on floor slabs was cut off in the centers of the spans between girders, generally a strip 20 ft. wide across the wharf being carried at one operation.

Very little patching was found necessary when the forms were stripped. The 6 by 8-in. braces were the members most requiring this attention. The tops of the shells, with the contained concrete, were rounded neatly by hand about the foot of the columns, and the entire concrete surface was coated with thick cement grout applied with brushes. Several persons (laymen) paid the compliment of inquiring what kind of wood was used for veneering.

All the concrete was conveyed in Ransome buggies from the hoist delivery, involving a maximum carriage of 400 ft.

Steel Assembling.—The system of reinforcement being that of "loose rods," much care was required in setting the steel and maintaining it in place. In comparison with the vigilance required in placing the concrete, this exaction was not severe.

The rods were shaped with a simple hand-bending lever worked on a bench, the angles desired being chalked on the top thereof. The cost per pound for cutting, bending and assembling was $0.0028. The

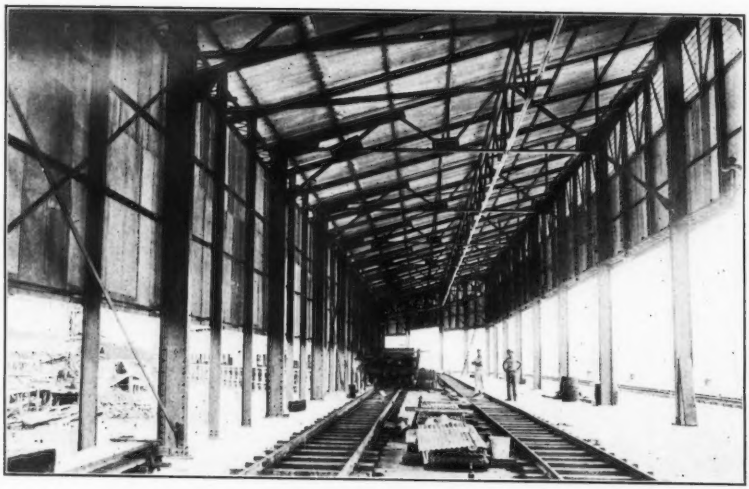

FIG. 1.—LOOKING EAST ALONG ALMIRANTE WHARF.

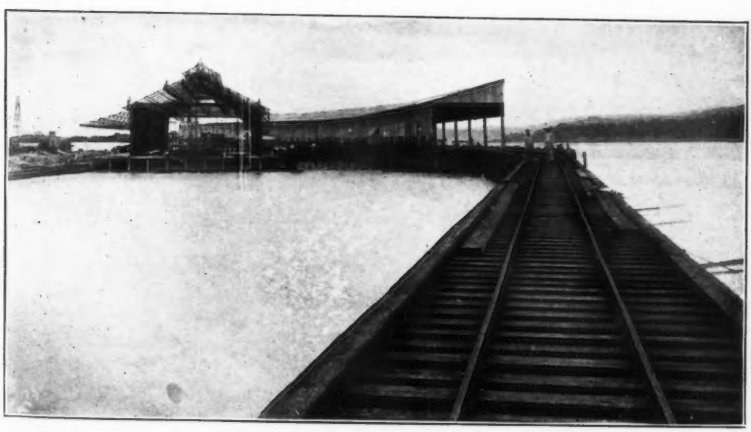

FIG. 2.—ALMIRANTE WHARF: LOOKING EAST FROM TRESTLE APPROACH, SHOWING WAREHOUSE, SHED, ETC.

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total weight of the reinforcement was 250 200 lb., exclusive of that in the shells, which was 2 700 lb., or 6¾ lb. per sq. ft. of wharf area, and amounted to 1½% of the volume of the concrete calculated above the level of the shell tops. The steel was furnished by the Baltimore Bridge Company.

Superstructure.—The shed over the fruit tracks extends the full length of the wharf, less 20 ft. at each end. The width is 30 ft. from center to center of the columns, the latter having the same spacing longitudinally. The frame is of steel, and was roofed with No. 20 galvanized-iron sheets; portions of the sides were covered with No. 22 sheets of the same material, as indicated on Plate LXVIII, which also shows the shape of the structure.

The weight of the structural work was 308 000 lb. The cost for distributing, straightening, and erection was $2 300, or $14.91 per ton of 2 000 lb. The paint and painting (two coats) cost $991, or $6.44 per ton. The labor on the covering, including the louvers, cost $939, exclusive of the painting.

Warehouse.—Of interest allied to that of the wharf are some of the points of construction involved in the warehouse and water supply. The warehouse is 50 by 200 ft. on the column lines. In addition to this, there is a platform 14 ft. wide along the service railroad, over which there is a structural steel roof extending to the center of this track. There is also a platform 9 ft. wide at the west end of the warehouse, and from this extends a bridge connecting with the wharf and serving for pedestrian and light trucking uses.

The warehouse foundation is on piles. A mass of concrete with roundabout reinforcement was used to form a head and base on each pile. The grade of the floor is 6 ft. above mean water, and the available space between that and the natural surface (Elevation 0) was utilized for a reservoir by building a floor, finished at Elevation 1, and a walled enclosure. The reservoir is divided into two compartments which may be used alternately or in combination, the total capacity being 300 000 gal.

The present supply for this reservoir is the roof-water from the warehouse, with the possibility of admitting at any time that from the wharf shed. The water is conducted about the yard in pipes of liberal size, and will serve as the potable supply. The necessity for pumping

the water is expected to prevent waste. The inner surfaces of the reservoir were treated with a soap and alum wash.

In commending the appearance of this structure, the writer need make no apology, as it was built mainly during his absence. To the credit of the superintendent and others contributing, it must be said that the integrity and finish of the concrete is second to none.

Water Supply.—A supply of water under pressure, for boiler and laundry uses, was obtained from a creek 2 miles inland. A small storage reservoir was provided on a hill, near the point of taking, and about 100 ft. above sea level. This reservoir is about 200 ft. from the line of the main leading to the wharf. It is of reinforced concrete, and is circular in form, having 6-in. walls, and is covered with a dome 4 in. thick. Its internal diameter is 30 ft., and it is 12 ft. deep, affording a storage of substantially 60 000 gal.

The interior surfaces were treated with a wash of soap and alum. There was no seepage when the reservoir was filled, which speaks well for the integrity of the work.

The main is of galvanized wrought-iron, having a diameter of 5 in. from the pump to the reservoir, and of 4 in. from the reservoir branch to the wharf, along which it is continued, with a branch to the warehouse. There are abundant outlets for service to ships as well as for the terminal structures, railroad yard, etc. Whenever the ordinary consumption becomes a serious factor in producing friction in the main, a small local storage capacity at the terminal will be of advantage.

Statistics.—

First pile driven, August 16th, 1907.
Last pile driven, March 27th, 1908.
First concrete poured, November 15th, 1907.
Floor finished, April 3d, 1908.
Area of the wharf floor = 37 160 sq. ft.
Number of piles in wharf = 432.
Number of shells in wharf = 432.
Average length of piles = 58 ft.
Average length of shells = 18.4 ft.
Quantity of concrete in shells = 7 230 cu. ft.
Quantity of concrete in filling shells = 6 180 cu. ft.
Quantity of concrete in columns, etc., = 35 100 cu. ft.

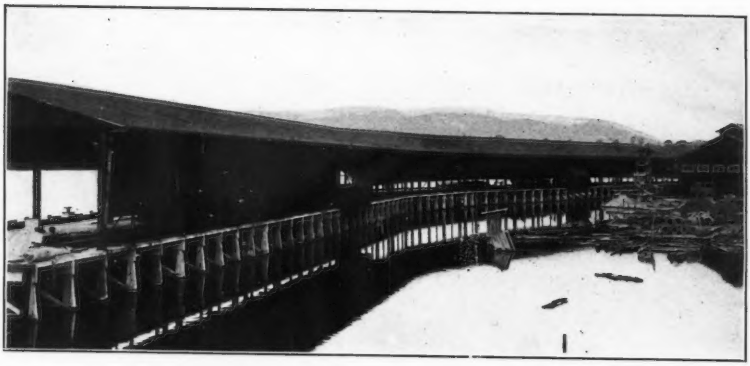

FIG. 1.—REAR VIEW, ALMIRANTE WHARF, LOOKING WEST.

FIG. 2.—REINFORCED CONCRETE RESERVOIR AND PIPE LINE.

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Cost of wharf floor	$79 377
Cost of approach	15 836
Cost of bridge to warehouse	608
Cost of fender	2 273
Cost of shed	26 770
Cost per square foot for:	
Labor for mixing and placing concrete	$0.08
Labor in making, erection, and removal of forms	0.204
Materials for 6 500 sq. ft. of forms	0.454
Same into area of wharf	0.08
Wharf floor, exclusive of shed	2.13
Cost per linear foot of shell	1.78
Cost of average shell	33.40
Cost per square foot of shed, structural work	0.94
Total cost per square foot of shed	1.38
Cost per 1 000 ft. of lumber on approach	25.90

The cost of general items, in which are included engineering, superintendence, watchmen, cooks, storage, and general supplies, was 24.2% of the other expenses, and is included in the costs above given.

General.

Labor.—The labor was almost entirely recruited from Jamaican negroes resident in the vicinity. There were also some Central American Spaniards and some Patois, but almost all were of African blood, or had a considerable mixture thereof. The Spaniards had the least African blood, and were the most intelligent, while the Patois were the least desirable, with the Jamaicans close competitors. In indolence, both the latter are hard to outclass, being also slow-moving to an exasperating degree. It is necessary to bear in mind constantly this limitation, apparently incurable, in order not to lose one's nervous balance.

The regular wages were $1.00 gold for 9 hours' work increasing to a maximum of $1.50, which was paid to riveters, so rated, but not experts. The average was about $1.20. In point of service rendered, the $1.00 man was worth say 40% of the value of an ordinary Italian (not an experienced one), which would make the latter figure $2.50 per day.

Quality of Work.—Credit must be given to the General Superintendent, Mr. Robert V. O'Brien, and to the Assistant Engineer, Mr. Chester S. Allen, for securing a quality of concrete, both in integrity and external finish, very gratifying to the writer. The foregoing remarks about the class of labor available will suffice to explain the difficulties under which the work was accomplished. Mr. Alphonse Briffod was in charge of the pile-driving, and Mr. F. Herbert Mallett was foreman of form making, erection, and removal, and ably seconded the efforts to achieve these results.

Remarks on Forms.—The writer would emphasize the value of regularity in forms. In the statement of costs that of 8 cents per sq. ft. for the labor in mixing and placing concrete may be compared with 20.4 cents per sq. ft. for the labor in making, erecting, and removing the forms. This is a significant contrast. In the case in hand there existed a section 100 ft. in length, comprising the central and widest portion of the wharf, where regularity could not be maintained. The irregularity in the positions of the railroad beams also served to retard the rapid erection of the forms. These beams might have been used on regular spacings had it been practicable to give the floor slab sufficient strength to take the concentrated locomotive load in whatsoever point. In two designs which the writer has in hand at present, it is possible to design the slab in this way with economy of materials, and without imposing too great a dead weight.

In conclusion, a word of caution should be given: In construction of this kind care should be taken to extend the protective shell to a depth secure against any future erosion and consequent exposure of the wood. Recent examples of the destruction of even creosoted piles by sea life at depths of 25 ft. emphasize this caution.

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AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

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THE OUTLET CONTROL OF LITTLE BEAR VALLEY RESERVOIR.

By F. E. Trask, M. Am. Soc. C. E.

To be Presented November 17th, 1909.

The second largest development project in Southern California is that of the Arrowhead Reservoir and Power Company, on the headwaters of the Mojave River, on the north slope of the San Bernardino Mountains and about 13 miles north from San Bernardino, Cal. In this paper the writer will present only one of the minor features of this project, reserving the consideration of the dams, tunnels, conduits, and power plants for some future paper, when these works are nearer completion.

Little Bear Valley Impounding Reservoir is being formed in a mountain valley by the construction of an earthen dam at the narrows in the Little Bear Valley Creek. The water-shed above the dam has a drainage area of 6.62 sq. miles. The dam will have a height of 200 ft., and will impound 60 000 acre-ft., or its equivalent, 19 551 000 000 gal. The supply of water will be largely augmented from adjoining drainage areas, by diversion works at such elevations as to permit the delivery of flood waters through tunnels into Little Bear Valley Reservoir.

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

Outlet.—The general direction of flow of Little Bear Creek is eastward, the dam being at the eastern margin of the reservoir. The outlet is through a tunnel leading north through the mountains, and about ½ mile west of the dam. Fig. 1, Plate LXXV, shows the relative positions of the dam and the outlet tower.

Outlet Control.—The outlet tunnel plans and details of control were designed and partly completed some years prior to construction work on the present dam. Fig. 1 shows the original design for the regulation and control of the discharge waters from this reservoir. In 1906 it became necessary to complete the outlet works, and the engineering department, after much deliberation, concluded to finish the work substantially in accordance with this design, although it was regarded as a departure from standard practice and likely to prove a failure. This procedure was justified because of the fact that the cost of so doing was the installation of equipment already in stock and would add but little to the expenses of the final structure; and the further reason that this design could be subjected to one season's working conditions without delaying or jeopardizing the work on the dam.

The work was completed and the gates were closed during the winter of 1906-07. Before the close of the rainy season, in the spring of 1907, the water level had reached an elevation about 20 ft. above the tunnel portal, and as any further rise of the water would endanger the partly completed dam, the gates were opened sufficiently to maintain this stage of the water.

Trouble immediately began, from the clogging of the screens installed at the outlet portal. During storms, several men armed with grappling hooks were kept busy in boats over the portal, and it was with the greatest difficulty that the screens were kept sufficiently clean to maintain the water level below the danger point.

At the close of the rainy season the water in the reservoir was drawn down to the level of the outlet tunnel, and all accumulated drift was cleaned from the screens. An examination of the portal disclosed the fact that the metal-work of the screen chamber had been seriously bent and distorted, and that the masonry had been ruptured and partly destroyed. This one season's test demonstrated that a screened reservoir outlet chamber—which, with a full reservoir, would be submerged under 165 ft. of water and located in a lake 500 ft. from

Fig. 1.

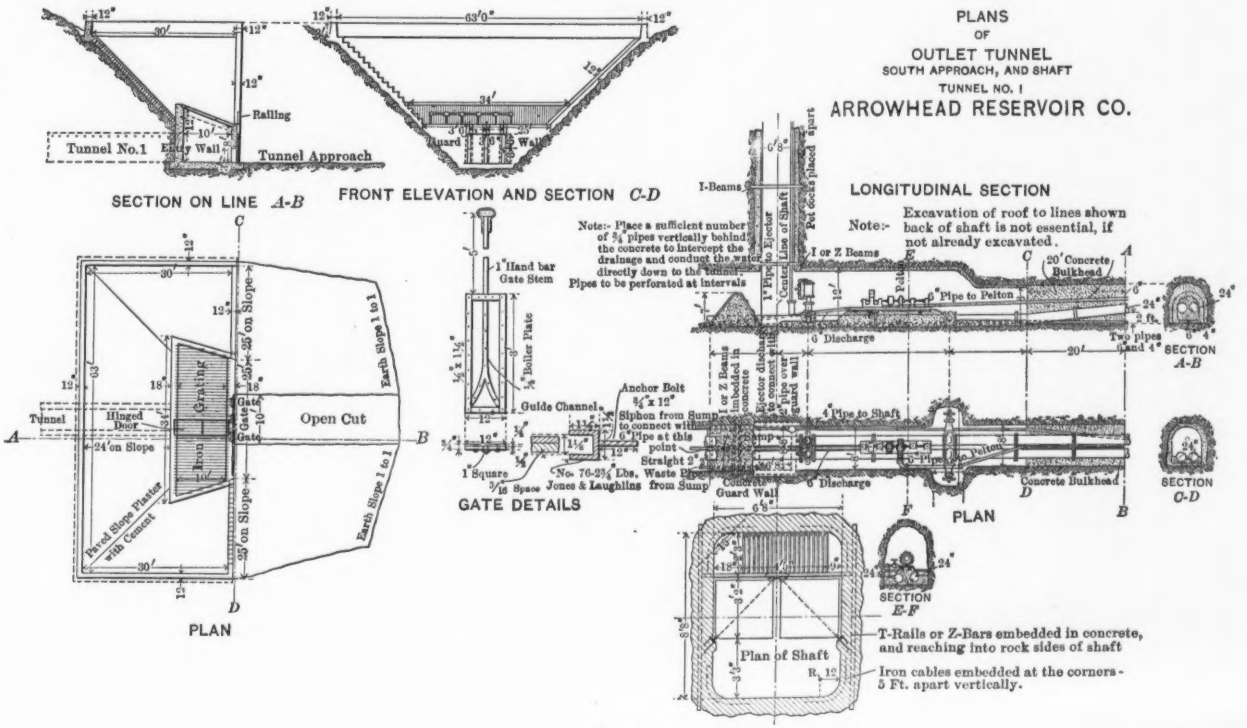
PLANS
OF
OUTLET TUNNEL
SOUTH APPROACH, AND SHAFT
TUNNEL NO. 1
ARROWHEAD RESERVOIR CO.
SECTION ON LINE A-B
FRONT ELEVATION AND SECTION C-D
LONGITUDINAL SECTION
PLAN
GATE DETAILS
PLAN
Tunnel No.1
Tunnel Approach
Railing
Open Cut
Iron Grating
Hinged Door
Tunnel
24' on Slope
Earth Slope 1 to 1
Pave'd Slope Plaster with Cement
1" Hand bar Gate Stem
1/8" Boiler Plate
Guide Channel
Anchor Bolt
Siphon from Sump to connect with
6" Pipe at this point
Straight 2" Jones & Laughlins
No. 76-2¾ Lbs. Waste Pipe from Sump
1" Square
3/16 Space
Note:- Place a sufficient number of ¾" pipes vertically behind the concrete to intercept the drainage and conduct the water directly down to the tunnel. Pipes to be perforated at intervals
Note:- Excavation of roof to lines shown back of shaft is not essential, if not already excavated.
I-Beams
I or Z Beams
Pet docks placed 8 feet
1" Pipe to Ejector
Center Line of Shaft
20' Concrete Bulkhead
6" Pipe to Pelton
6" Discharge
Two pipes 6 and 4"
SECTION A-B
4" Pipe to Shaft
Pelton
Concrete Bulkhead
SECTION C-D
Guard Wall
SECTION E-F
Plan of Shaft
T-Rails or Z-Bars embedded in concrete, and reaching into rock sides of shaft
Iron cables embedded at the corners - 5 Ft. apart vertically.

FIG. 1.

its nearest shore margin—was a dangerous innovation, not to be retained in any mountain flood-water storage reservoir.

Reinforced Concrete Outlet Tower.—As a substitute for this condemned structure, it was decided to build a reinforced concrete screen, and gate-tower, near the portal of the tunnel, and connect it with the gate-shaft by a reinforced concrete conduit placed in the outlet tunnel. Fig. 2 shows the general features of the tower, conduit, tunnel, and gate-shaft, and includes the essential details of the tower and the conduit. Fig. 1, Plate LXXV, shows the tower completed and the water in the reservoir standing at a depth of about 20 ft. over the tunnel portal. The photograph, Fig. 2, Plate LXXV, was taken at the time the reinforcing steel for the base and the first section of the tower was set and ready for placing the concrete. Fig. 3, Plate LXXV, shows the tower about half completed, and illustrates the method of handling the material during construction.

A contract for the work was let, the company supplying cement, valves, castings, reinforcing steel, and all metal, and the contractor furnishing all labor, tools, rock, and sand, the contractor receiving and placing all metal parts.

The specifications required that the crushed rock should pass a screen having a 1-in. mesh and that the sand used should be from the creek bed or crushed from live rock and passed through a screen having a $\frac{3}{16}$-in. mesh. The relative percentages in the concrete mixture were subject to the regulation of the engineer, and as used gave the proportions: cement 1, sand 3, and rock 2. These proportions produced quite impermeable concrete when thoroughly spaded and tamped into the walls of the tower. Special attention was directed to bonding the joints between the work of succeeding days; the whole contact surface was chipped off and a groove 2 in. deep was cut on a radius 8 in. less than that of the outside of the tower. The whole surface then received a thorough washing, followed by a coat of cement grout, after which the next section of tower was placed.

The contractor carried on the work during the winter season, and provided against frost action by covering the top of the tower with canvas and maintaining a fire in the base during freezing weather. Under these conditions the progress was slow, as the concrete could be placed only when the weather was favorable.

The following figures are from the engineer's notes, and give the

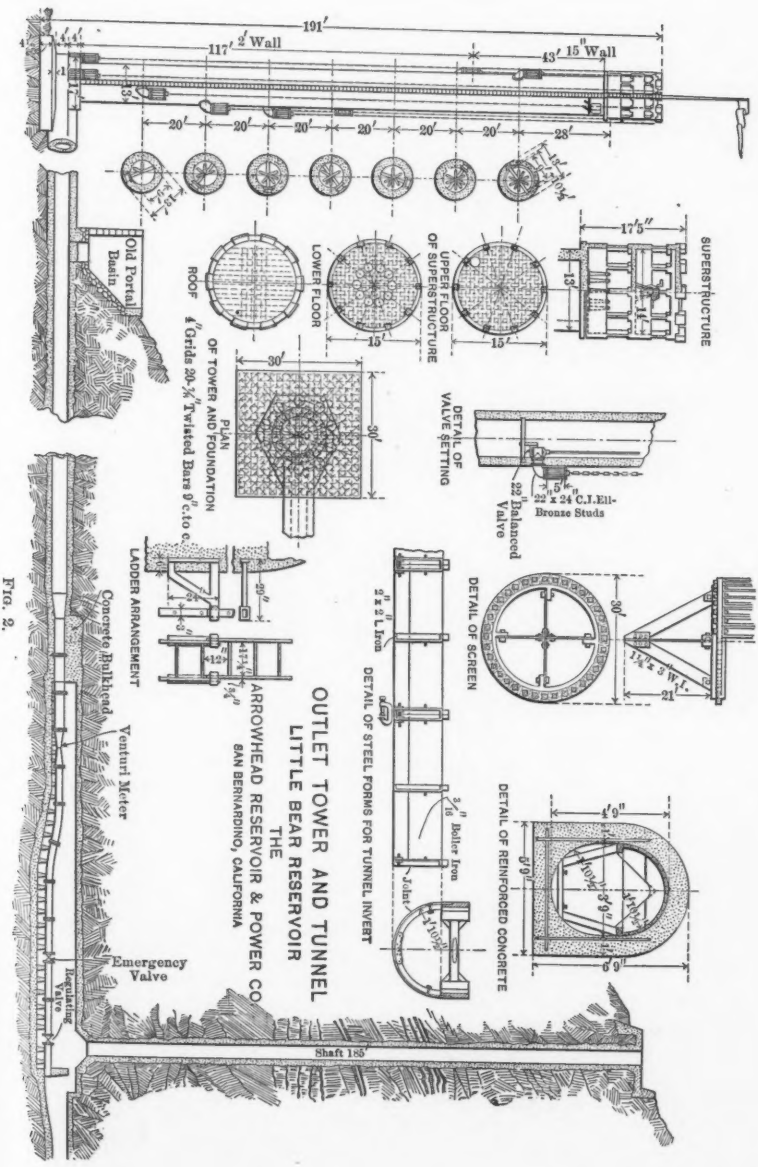
191′
117′ 2′ Wall
43′ 15″ Wall
20′
20′
20′
20′
20′
20′
28′
Old Portal
Basin
ROOF
LOWER FLOOR
UPPER FLOOR OF SUPERSTRUCTURE
SUPERSTRUCTURE
17′5″
13′
15′
15′
30′
30′
PLAN OF TOWER AND FOUNDATION
4″ Grids 20-⅞″ Twisted Bars 9″ c. to c.
DETAIL OF VALVE SETTING
22″ Balanced Valve
22 x 24 C.I. Ell-Bronze Studs
LADDER ARRANGEMENT
20″
12″
DETAIL OF SCREEN
30″
21′
DETAIL OF STEEL FORMS FOR TUNNEL INVERT
Boiler Iron
Joist
DETAIL OF REINFORCED CONCRETE
4′0″
6′0″
6′0″
Concrete Bulkhead
Venturi Meter
Emergency Valve
Regulating Valve
Shaft 185′
OUTLET TOWER AND TUNNEL
LITTLE BEAR RESERVOIR
THE ARROWHEAD RESERVOIR & POWER CO
SAN BERNARDINO, CALIFORNIA

FIG. 2.

cost per cubic yard for the labor of placing 12½ cu. yd. of concrete in the tower at an elevation 70 ft. above the portal:

6 mixers	at	$2.50	$15.00
2 loaders	"	2.50	5.00
2 dumpers	"	2.62½	5.25
2 carpenters (tamping)	"	3.75	7.50
1 tamper	"	2.50	2.50
1 engineer	"	3.50	3.50
Engine rent			3.00
Fuel			2.50
Total cost			$44.25
Total cost per cubic yard			$3.58

These figures are much less than the average cost to the contractor, owing to the many delays and incidentals.

Design.—The problem of outlet regulation, presented to the engineers, required no new or radical departure from standard practice, and when the old work failed it was quickly decided to build a tower of reinforced concrete, with intake openings each 20 ft. in elevation, and equip each with a balanced valve and screen, the valves to be operated within the tower, from a house upon its top, the screens to be operated from the house, but outside of the tower.

The design further provided for connecting the tower with the outlet tunnel by a reinforced concrete conduit of such design and strength as to resist the crushing strains, when empty, resulting from a full reservoir with 160 ft. of water above the conduit. A gate control in the gate-shaft, in the original plans, will be retained, and the balanced valves in the tower will be used as emergency gates.

Sufficient material was put into this monolithic structure to give it gravity stability with a full reservoir, and the twisted steel rods incorporated were calculated to take all tension strains from the masonry, on a basis of 16 000 lb. per sq. in. of metal.

The resistance to overturning was computed on a basis of an empty reservoir and a wind velocity of 80 miles per hour. The formula adopted was for beams fixed at one end, with a uniform load; and the tower was analyzed for these and all other possible conditions and strains at each 10-ft. section from the base up.

FIRST SECTION OF TOWER.

DURING CONSTRUCTION OF TOWER.

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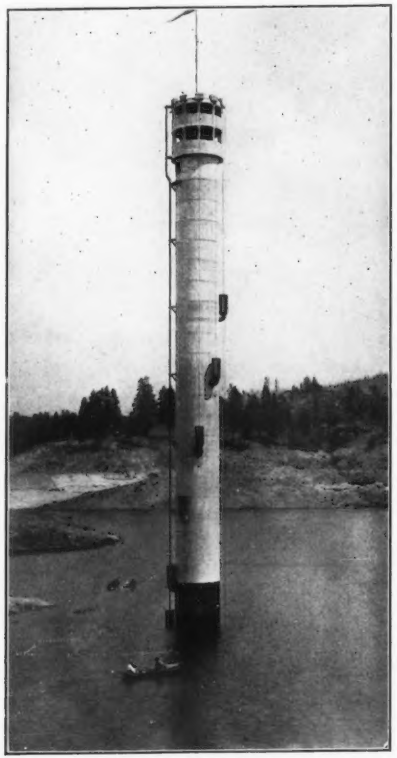


FIG. 1.—COMPLETED TOWER OF LITTLE BEAR VALLEY RESERVOIR.

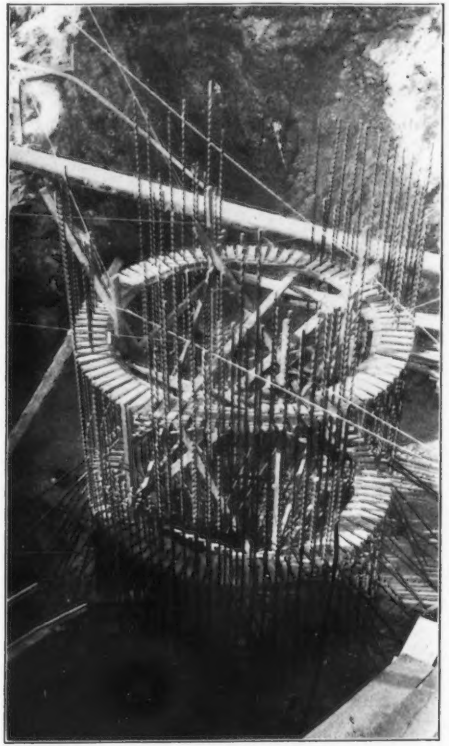


FIG. 2.—REINFORCING STEEL IN PASE AND FIRST SECTION OF TOWER.

FIG. 3.—METHOD OF HANDLING MATERIALS DURING CONSTRUCTION OF TOWER.

The tower was built by Mr. A. S. Bent, of Los Angeles, a contractor of much experience in concrete work. Mr. Walter Hy. Brown, Assistant Engineer (now Acting Chief Engineer), had charge of working out the details of the tower and of supervising its construction. The high class of work obtained is largely due to his untiring efforts and study.

The Consulting Engineers of the Arrowhead Reservoir and Power Company, Mr. F. C. Finkle, and the writer, of Los Angeles, Cal., are responsible for the general design of this structure, and have kept a general supervision over its construction.

Vol. XXXV. SEPTEMBER, 1909. No. 7.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

HYDRO-ELECTRIC POWER IN CANADA.
Discussion.*

By H. Holgate, M. Am. Soc. C. E.

Mr. Holgate.

H. Holgate, M. Am. Soc. C. E. (by letter).—This subject is one of the great features of present-day development and advancement, and is of growing importance and interest to the community.

The well-known phrase "Conservation of Natural Resources" can probably be applied to hydraulic power development along lines similar to those which apply to forests and their preservation or destruction. A forest may be wasted or burned, but the land, in most cases, will reproduce something of value. In developing hydraulic power, a permanent condition of affairs may be created, so that the full potentiality of the river at the development site can never be realized, thus causing a permanent waste of power. Of course, there are commercial features which sometimes limit the capital outlay and prevent the building of works to produce maximum results, but, it often happens that permanent works are built for present requirements only, and are afterward extended or improved to meet the growth of business, generally resulting in a greater outlay than would have been necessary originally, and also, even then, owing to the limits imposed by initial design, failing to be able to utilize all the power which might be available, or even may be required.

No water-power proposition should be undertaken before it has received the fullest possible study; and investors would be consulting their own interests if they secured sufficient information to establish the total physical possibilities, and the cost of the development. It is

* This discussion (of the paper by Cecil B. Smith, M. Am. Soc. C. E., printed in *Proceedings* for May, 1909, and presented at the meeting of September 1st, 1909), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

of first importance to establish the full potentiality of the stream, making use of all favorable physical features; then any modifications which business prudence may dictate may be made, but one should not lose sight of the fact that at a future time it may become necessary to expand and carry out an ideal form of development; therefore, the earlier works should be built to fit in with future work, as far as practicable, and thus avoid unnecessary expense and costly interruption of business. Mr. Holgate

It is not possible to say what is to be in the future, but lack of confidence in the future progress of Canada has stamped itself on many a hydraulic power development, and the time has come when all doubts and fears for the great expansion of trade in the Dominion may be dismissed; Canada may now be said to have emerged from the "hobbledehoy" stage of growth and to have entered upon good sturdy progresssive manhood, with growing confidence in the future.

In granting water-power rights, it will be proper for the Government to insist that the initial works shall be built with due regard to the possible maximum development in the future, so that all possible power can be made available when conditions arise which may demand its use.

Though Mr. Smith's paper does not contemplate going into details, yet he has mentioned two plants which were designed and built by the writer and are used extensively for mining and smelting purposes. The first is the development at Bonnington Falls, on the Kootenay River, in British Columbia, and the second is the development for the International Nickel Company at High Falls, on the Spanish River. The latter serves all the works of the International Nickel Company in what is known as the Sudbury District, in Ontario.

As these two plants are radically different in design, and represent different methods of development, their cross-sections, Figs. 6 and 7, may prove interesting.

The water-powers of Canada are distributed over almost the whole country, but the district east of Port Arthur is particularly favored, and, owing to the accessibility of the powers south of the height of land, those on rivers tributary to the St. Lawrance will naturally be developed before those on rivers flowing into Hudson Bay (erroneously called Hudson Sea).

In particular, attention is called to the possibilities of the St. Lawrence, and to the rivers tributary to it east of Quebec, as this territory, in so far as the north shore of the St. Lawrence is concerned, is not referred to in Mr. Smith's paper, nor has it received much attention from any one.

In regard to the St. Lawrence River, the mean level of Lake Ontario is 245 and the mean level of the St. Lawrence at Montreal is 18, thus creating a fall of about 227 ft. The minimum flow of the river

Mr. Holgate. is about 187 000 cu. ft. per sec., so that, on this basis, the minimum energy of the river in this stretch in about 100 miles is about 3 500 000 h.p.

The fall in the river is overcome by a canal system affording a depth of 14 ft., with locks 255 by 45 ft. The various canals are:

Canal.	Lockage.	Length.
Galops	17.81 ft.	7.60 miles.
Morrisburg	14.81 "	3.70 "
Farren's Point	10.78 "	0.75 "
Cornwall	46.82 "	11.50 "
Soulanges	84.13 "	14.00 "
Lachine	47.62 "	8.50 "
Totals	221.97 ft.	46.05 miles.

This leaves stretches of river navigation aggregating about 54 miles, so that nearly one-half of the navigation distance from Lake Ontario to Montreal is through the canals.

The present depth of the canals will not carry the larger Lake boats, and the question of deepening the canals and enlarging the locks is being discussed. This would be a very heavy undertaking, and would result practically in rebuilding all the canal works, as the new depth should be at least 22 ft.

An exhaustive investigation into the feasibility of damming the St. Lawrence at the foot of the Long Sault Rapids, a few miles west of Cornwall, has been carried on by J. W. Rickey, M. Am. Soc. C. E., during the past three years, and the writer has examined all the data gathered, and has no doubt of the possibility of carrying out the work. The result of building dams at the sites selected will be the total elimination of floods above the rapids, caused now by ice jams, and the improvement of flood conditions due to ice below the rapids, and, in addition to this, the present Cornwall Canal could be eliminated by placing a single lock at the dam.

Ice conditions on the St. Lawrence are annually a source of trouble, and, of course, this would be prevented by treating the rapids in this manner. The writer is convinced that a careful study will show that it is possible to dam the river at the foot of each of the rapids, thus providing for navigation throughout the whole distance, and using the present canal system as an alternative route. In doing this, the necessary depth could be obtained, and navigation conditions would be decidedly improved by providing a deeper channel and by permitting vessels to move with increased speed, as they must now proceed through long stretches of canal with numerous small lockages, making their progress very slow, both up and down.

Of course, the tourist will complain that he cannot "shoot the

Mr. Holgate.

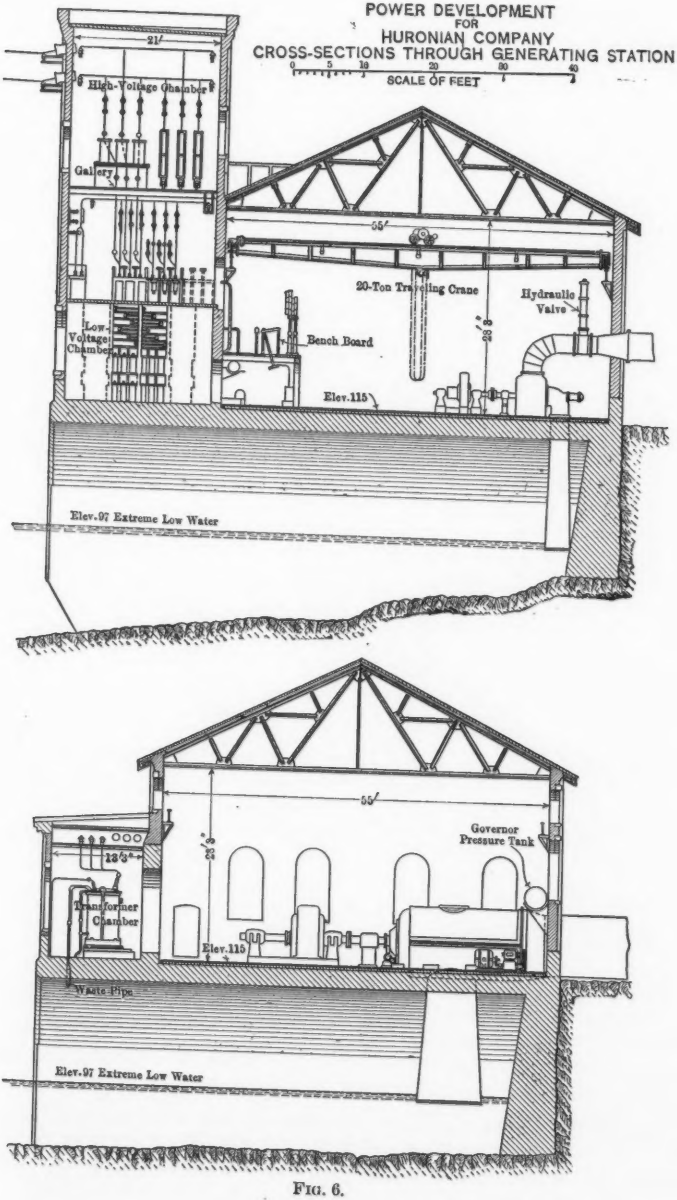


FIG. 6.

Mr. Holgate. rapids," and that the natural beauty of the river is marred, but when it is considered that carrying out such a work will provide a vast amount of power, and also will improve a great international water highway, the argument of the lover of wild Nature can have little weight.

In some cases the construction of such works would not be justified if the result were only the production of power, but coupled with this there is the possible improvement to navigation, and the two results will justify at least an exhaustive study of the river, with this end in view; particularly is this so when the increasing value of power is considered, and also the increasing importance of this water route.

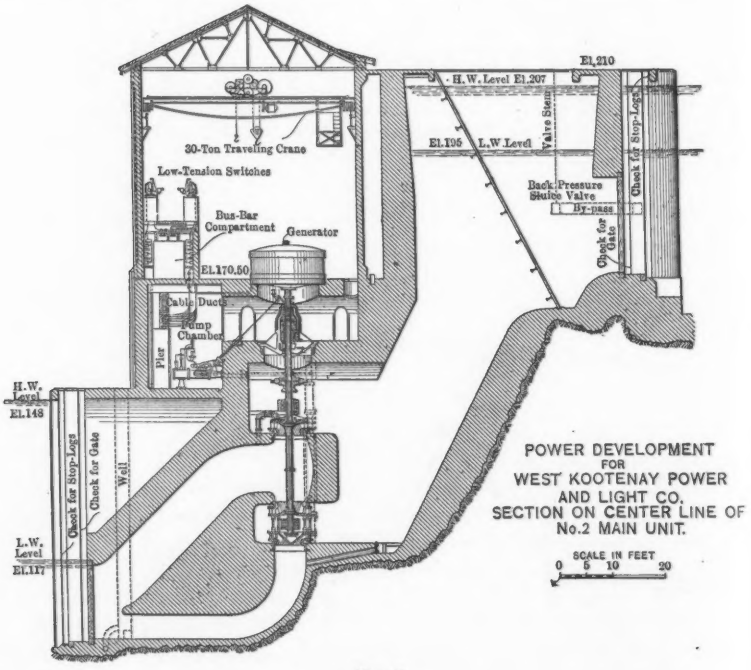
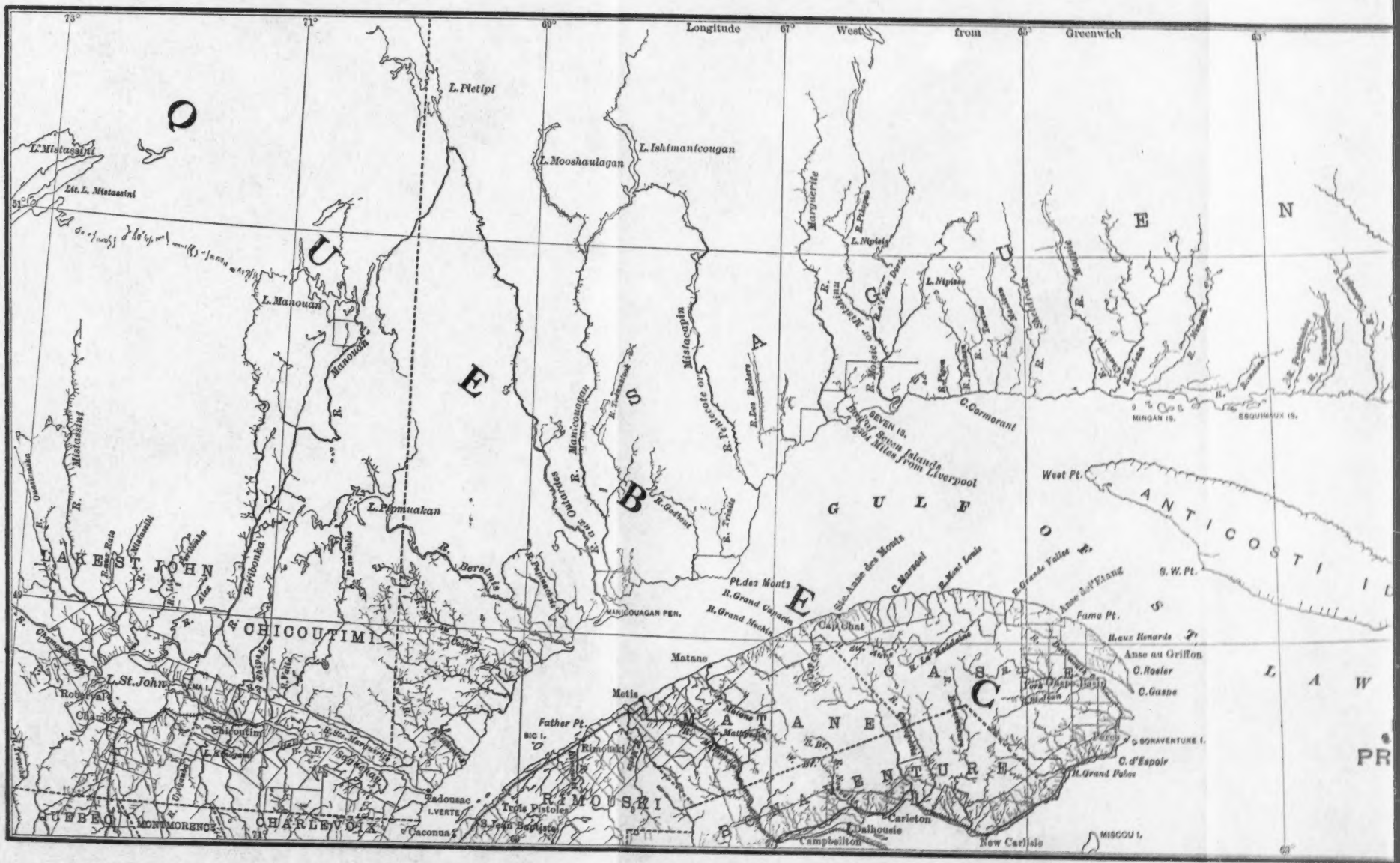


FIG. 7.

The relations between the Nations are happily such as to render it possible to bring about a solution of the problem, and if they co-operate, the St. Lawrence may be made a gigantic power producer, and a more important water highway than it is now.

Attention is next drawn to the possibilities offered for the development of water-power east of Quebec, on the rivers flowing south into the St. Lawrence. These begin with the Saguenay and may be said to end with the Natisquahan River, although there are gigantic water-


Longitude West from Greenwich
L. Pletipi
L. Mistassini
Lit. L. Mistassini
L. Mooshaulagan
L. Ishimanicougan
L. Manouan
Manouan R.
L. Pipmuakan
R. Bersimis
R. Ouiatchouan
R. Mistassini
R. Peribonka
L. St. John
LAKE ST JOHN
CHICOUTIMI
Chicoutimi
Roberval
Chambord
Ste. Anne
Metis
Matane
Father Pt.
Rimouski
Trois Pistoles
Tadousac
I. VERTE
Caconna
St. John Baptiste
QUEBEC
MONTMORENCY
CHARLEVOIX
R. Manicouagan
R. Outardes
R. Godbout
R. Trinité
R. Mistacapin
R. Ste. Marguerite
L. Nipisis
R. Moisie
C. Cormorant
Seven Islands
Distance 3051 Miles from Liverpool
MINGAN IS.
ESQUIMAUX IS.
GULF OF ST LAWRENCE
MANICOUAGAN PEN.
Pt. des Monts
R. Grand Capadoc
R. Grand Mecatina
Ste. Anne des Monts
C. Rosier
C. Gaspe
Cape Chat
Anse au Griffon
Fame Pt.
West Pt.
S. W. Pt.
ANTICOSTI I.
BONAVENTURE I.
C. d'Espoir
Grand Pabos
Carleton
Dalhousie
Campbellton
New Carlisle
MISCOU I.
PRINCE EDWARD I.

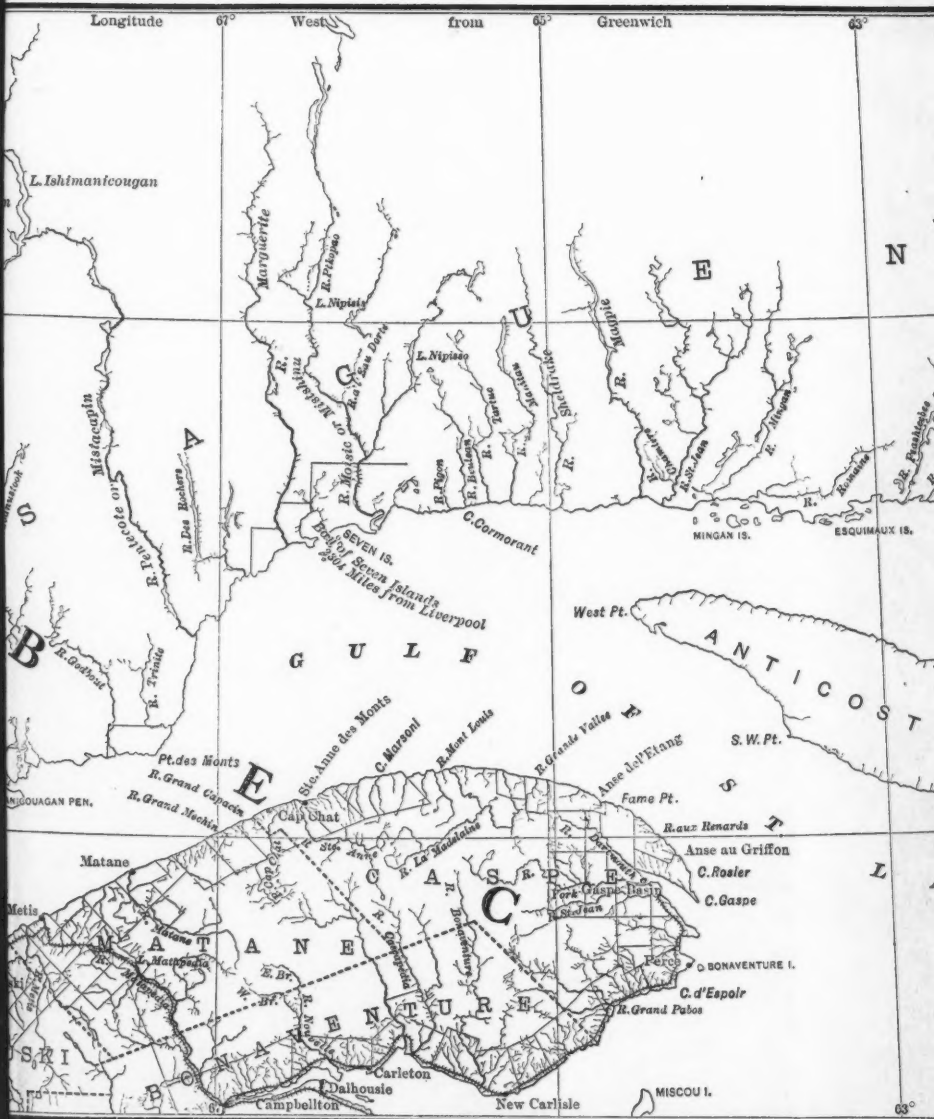
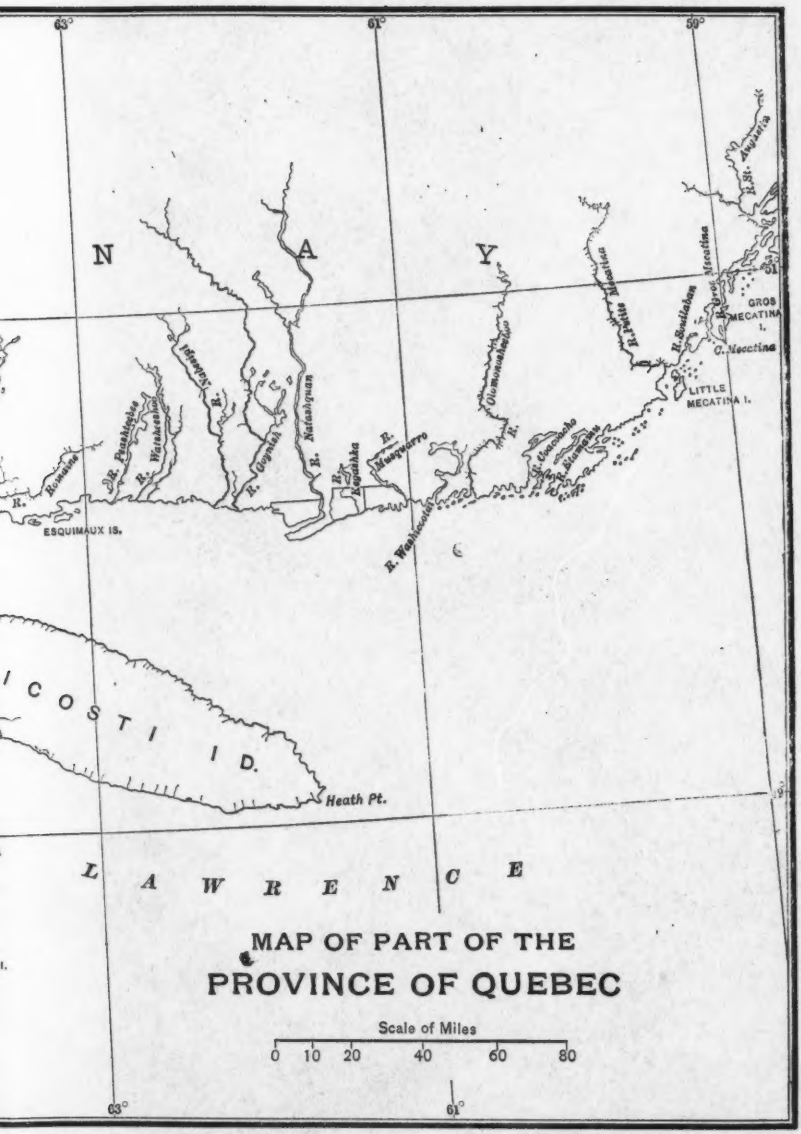

Longitude 67° West from Greenwich 63°
L. Ishimanicougan
Marguerite
R. Pitapago
L. Nipisis
R. Watshishou
R. aux Eaux Mortes
L. Nipissis
R. Manitou
R. Pigou
R. Bonsens
R. Tarsis
R. Jupiter
R. Shecatica
R. Munnie
R. Romaine
R. Mingan
MINGAN IS.
ESQUIMAUX IS.
Mistacapin
R. Des Rochers
R. Godbout
R. Trinité
C. Cormorant
SEVEN IS.
Depth of Seven Islands 2504 Miles from Liverpool
West Pt.
ANTICOSTI
S. W. Pt.
Fame Pt.
R. aux Renards
Anse au Griffon
C. Rosier
C. Gaspe
Anse de l'Etang
R. Grande Vallée
C. Marconi
R. Mont Louis
Ste. Anne des Monts
Pt. des Monts
R. Grand Capacka
R. Grand Machin
Cap Chat
R. Ste. Anne
R. La Madeleine
Matane
Metis
R. Matane
L. Matapedia
R. Matapedia
R. Restigouche
R. Patapedia
Fork Gaspé Basin
Ste. Jean
Perce
BONAVENTURE I.
C. d'Espoir
R. Grand Pabos
Carleton
Dalhousie
Campbellton
New Carlisle
MISCOU I.
63°

PLATE LXXVI.
PAPERS, AM. SOC. C. E.
SEPTEMBER, 1909.
HOLGATE ON
HYDRO-ELECTRIC POWER IN CANADA.



UNIV. OF MICH.

powers on the Hamilton River, which, at present, are commercially inaccessible. Mr. Holgate.

Large developments are possible on the Saguenay, Bersemis, Godbout, Outarde, Manicouagan, Manitou, Romaine, St. John, and Natisquahan Rivers. A development of power on St. Marguerite River, made by the writer, is used for the manufacture of wood pulp.

An unique situation occurs about 200 miles east of Quebec, where the Outarde and Manicouagan Rivers enter the St. Lawrence about 5 miles apart. There, within a radius of 5 miles, the writer has examined three possible developments, the combined horse-power of which would be 600 000.

All the water-powers north of the St. Lawrence are close to tide water, the back country for a long distance is covered with spruce, and iron exists in large bodies, so that it is only a matter of time when these powers will be commercially utilized. When smelting by electricity becomes a pronounced possibility, these water-powers will become particularly useful; at present they offer inducement principally to the paper-making industry.

Winter conditions, of course, are severe in this district, but not so severe as to affect operation materially, and the larger the river, the less trouble will be experienced in the operation of the power.

Plate LXXVI, a map of this district, shows the location of the rivers, and it should be noted that the distance from these points to Europe is much less than from any of the Atlantic ports.

The water-power possibilities of Canada are widespread and very great, and under wise direction, with the rapid growth of the country, it is reasonable to expect a great advance in the utilization and development of hydraulic power. Though many sections of the country where important developments are in operation have not been mentioned, it will be seen that much has yet to be done before the resources of Nature are even partially made use of.

Electro-chemistry, requiring large quantities of power at low cost, will find a great field in the water-powers of Canada, and will no doubt assist in the early advancement of power development.

Vol. XXXV. SEPTEMBER, 1909. No. 7.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

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PROGRESS REPORT OF SPECIAL COMMITTEE ON CONCRETE AND REINFORCED CONCRETE.

Discussion.*

BY MESSRS. JOHN STEPHEN SEWELL, CLARENCE W. NOBLE, S. BENT RUSSELL, J. R. WORCESTER, AND W. J. DOUGLAS.

Mr. Sewell.

JOHN STEPHEN SEWELL, M. AM. SOC. C. E. (by letter).—The writer read this report when it was first published with a great deal of interest; but after studying it carefully, he is compelled to agree with the minority on practically every criticism offered. Most of the conclusions in the Majority Report are merely stated without demonstration of their correctness; the reasons for assumptions are not set forth. In a report of this kind, the members of the Society should be furnished with the data in order to enable them to judge of the correctness of the Committee's conclusions.

If this Society were a body of laymen, and the Special Committee had been retained as experts to formulate rules governing the use of concrete and reinforced concrete, the form of the Majority Report might be accepted as correct. But these conditions do not obtain, and the members of the Society are entitled to the data and the reasoning which have led to the conclusions. It might even be helpful if the deduction of the somewhat complicated formulas were also included.

The writer strongly concurs in the recommendation of the minority that the report be referred back to the Committee for revision.

The following are some of the points that seem worthy of further attention:

DESTRUCTIVE AGENCIES.

Common Salt.—There is good reason for believing that when common salt, or sea salt, is mixed with the concrete, either to prevent freezing, or by reason of the use of sand or gravel from the seashore,

* Continued from April, 1909, *Proceedings*.

or of sea water for mixing, steel embedded in the concrete will be corroded, especially if the concrete is in a damp location. It ought not to be difficult to determine the facts once for all. Mr. Sewell.

Manure.—Fresh stable manure may have an acid reaction, but the writer has recently seen statements from expert chemists that manure and urine are both alkaline. The chemists differed as to whether or not water that had filtered through manure- and urine-soaked ground would be deleterious to green concrete. If the members of the majority are in possession of positive evidence on these points, they should produce it.

Ensilage.—The writer has seen a well-hardened concrete silo badly pitted after one season's use for ensilage made of green cornstalks. As this use of concrete is likely to increase, such action should be investigated.

Oils.—The production of positive evidence as to the action of different oils, with a definite statement of the origin and composition of those having an injurious effect, would be of value. The statements in the report would not be of much assistance in practice.

Fire-proofing.

The treatment of concrete as a fire-resisting material by the Majority Report is inadequate. Igneous rock is not necessarily the best aggregate. It often spalls badly, and may be less desirable than limestone, notwithstanding the inevitable calcination of the latter. Broken bricks, broken terra cotta, gravel, furnace slag, and clinkers are all good aggregates for fire-resisting concrete—better probably than any kind of broken stone. Concrete not only suffers dehydration in a fire, but it often spalls in the same way as granite. The thicknesses recommended by the Majority Report for fire-resisting purposes are not sufficient to provide for spalling, and should be regarded as the minimum allowable under the most favorable circumstances. A higher standard should be set for more severe conditions. The desirability and possibility of freeing the fire-resisting covering—at least of columns—from structural stress, should receive consideration.

Reinforced Concrete.

It seems a little inconsistent to assume for the ratio, N, a value of 15 when designing the mid-section of a beam, and a value of 8 when computing the deflection of the same beam. If the methods, assumptions, and notations of the majority be adopted, it will be found that in designing rectangular beams when $f_s = 16\,000$, $f_c = 650$, and $N = 15$, $\frac{A}{bd} = 0.0077$. If the same beams be analyzed for stresses, it will be found that if $\frac{A}{bd} = 0.0077$, $f_s = 16\,000$, and $N = 8$, $f_c = 985$.

Mr. Sewell. There is little doubt that, for 1:2:4 concrete at an age of three months and upward, the true value of N is more nearly 8 than 15. 985 lb. per sq. in. seems high for f_c, yet no one doubts that the value of $\frac{A}{b\,d} = 0.0077$ is, in most cases, safe enough, even if it is intended to allow f_s to reach a value of 16 000 lb. per sq. in. It seems to the writer both simpler and more accurate to determine the maximum allowable percentages of reinforcement from tests, and then to design the mid-section without reference to either N or f_c.

The matter of web stresses and diagonal tension is very inadequately covered by the Majority Report. If it is unsafe to count on the tensile strength of the concrete to resist horizontal tension, it is equally unsafe to count on it for diagonal tension, or for shearing stresses in the web. The ultimate solution is to provide a complete system of inclined web members, rigidly attached to the horizontal reinforcement, and designed and spaced to take all the tensile web stresses; they should extend to the top of the beam and be designed so that their adhesion in the compressed part of the concrete will develop their full strength. The writer enunciated this principle, as a result of theoretical study and a few isolated experiments, more than five years ago. The recent experiments at the University of Illinois and the University of Wisconsin, in his judgment, have proven it correct. The recommendations of the Majority Report do not cover the ground, and do not establish a definite standard for the design of the web. Diagonal tension failures are the greatest danger to be feared in reinforced concrete work. Nearly every beam failure in practice has been of this kind. In the writer's judgment, the recommendations for the design of T-beams in the Majority Report would not suffice to avoid designs dangerously weak under the web stresses. Two layers of bars, one above the other, should not be allowed. A definite statement as to how the stress from the bars is to be transmitted through the rib and into the flange of a T-beam should be made a part of the report. The vague requirement for "effective bond" at the junction of rib and slab is not sufficient. A study of current practice will soon convince one that it is dangerously inadequate. A definite standard should be established. The effect of great depth, as compared with the span, in increasing the danger of failure under web stresses, should be treated at length.

The writer feels that recent experiments afford sufficient data for a much more positive stand than has been taken by the majority on this general subject of web stresses. Its position in some points is absolutely at variance with the results of some of these experiments. For instance, the tests reported in Bulletin No. 197, University of Wisconsin, show that loose stirrups are more efficient when inclined than when they are vertical. The requirement of the Majority Report

Mr. Sewell.

that inclined stirrups must be rigidly attached, while vertical ones may be loose, seems to demand explanation. It is a little hard to see, also, why the Majority Report selects 75° as the angle of inclination. The tests in Bulletin No. 197 indicate that rigidly attached web members, whether inclined or vertical, are more efficient than loose ones; and that rigidly attached members are much more efficient when inclined at an angle of 45° than when vertical.

The many advantages of rigidly attached web members have seemed so obvious to the writer that he has often wondered why so many engineers have refused to admit them. It has occurred to him that the reluctance to commit one's self to a patented article may have influenced many purely professional engineers to a greater or less extent. It may be of interest to such men to know that the writer has had opinions from at least three competent patent attorneys and one patent expert, that the broad principle of attached web members is public property, and no valid patent, except on particular methods of applying it, can be obtained, or is in existence. At least three methods of applying it have been patented by three different inventors, and the writer has applied it successfully and economically with commercial bars, in a way that is not patented, and which, in the opinion of at least one competent patent attorney, cannot be patented. However, even if the broad principle were covered by patents, if it is the true solution, it should be adopted, and the inventor should get his reward. The writer asssumes that the Corliss valve gear was once patented, but if so, mechanical engineers did not deny their clients the advantages of Corliss engines until the expiration of the patents enabled any one to make and sell them.

Columns.

The writer has not had time to investigate for himself the majority's conclusions with reference to these members. But, in any event, these conclusions should be supported by a discussion of the data from which they were derived. If banded columns are to be used, it would seem that the bands should be designed, where the ends come together, so as to develop the strength of the material. It would also seem that a column should not be considered as hooped or banded unless the hooping or banding is circular.

Footings.

Current practice affords many instances of vicious and inadequate design of footings. This subject demands attention quite as much as any other.

There are many other points, such as permissible heights of reinforced concrete buildings; splicing of steel bars in superimposed columns; details of design at juncture of columns, beams, and girders,

Mr. Sewell. to resist secondary stresses; wind-bracing; resistance to earthquakes, shock, and vibration; design of outer walls and partitions in reinforced concrete; and design of structures as a whole, which demand attention, and on which the present state of the art affords, at least, a reasonable amount of information.

The writer feels that the Majority Report does not do justice to the able personnel of the Committee, and sincerely trusts that it may be referred back and carefully reconsidered.

Mr. Noble. CLARENCE W. NOBLE, ASSOC. M. AM. SOC. C. E. (by letter).—The importance of the work of this Committee cannot be exaggerated. The Society has rarely, if ever, appointed a committee whose work has such a wide scope and is of such prime importance to the Engineering Profession.

The public expects that engineers will be governed in their work by two considerations: First, the consideration of safety, and, second, the consideration of economy.

In their anxiety to satisfy perfectly the first requirement, engineers must not overlook the fact that the second requirement must also be considered. If a design is safe, the addition of superfluous material does not increase its safety and does increase its cost. Skill in designing is the result of the application of adequate knowledge and of the use of material in such a manner as to insure economy as well as safety.

The consideration of economy must not be forgotten in the present discussion. Engineers now have a large amount of information regarding the stresses in a reinforced concrete beam when merely supported at the ends. They have a certain amount of information—and need considerably more—regarding the stresses in reinforced concrete beams when incorporated with others in floor panels in actual building construction. A series of carefully observed deflection tests taken on buildings which have been constructed and are in actual use would be of service.

At one time the writer had one of his designs submitted to such tests. He designed the reinforced concrete floors for a large denominational school. It happened that the architect did not belong to the same religious denomination as the school, and that he let the work at a cost plus percent. basis to contractors who did not belong to this denomination. Later it was decided to extend the building, and contractors of the same religious belief as the school made a desperate attempt to discredit the architect by attacking the strength of the reinforced concrete construction.

To do this they tested the floor slabs in seven different places, the tests ranging from two to four times the safe load and extending over a width as great as the length of the panel. The loads were placed so as to avoid arching, and deflections were measured by a simple

system of levers which multiplied the actual deflection by ten. The deflections of the adjacent beams were also noted, where they could be detected, and subtracted from the slab deflections. The results were entirely satisfactory from the writer's point of view, and the experience obtained was of considerable value. Mr. Noble.

These slabs were supported on the exterior brick walls of the building on corbels, immediately above which there was built a reveal about $2\frac{1}{2}$ in. deep. In some cases the interior supports consisted of reinforced concrete beams and in some cases of brick walls. All the bars were ordered long enough to lap well across the interior brick walls or concrete beams at the top of the slab, the intention being to build the concrete slabs across the walls and then continue the walls above them. The contractor, however, cut all the rods where they lapped across brick walls, and erected his brick work in advance of the concrete, using a corbel and reveal on the interior walls as well as on the exterior walls. All the slabs were designed by the same bending-moment formula, regardless of location. Some of the slabs tested were supported on brick walls at both ends, some on brick walls at only one end, and some between two reinforced concrete beams.

While the deflections noted could not be actually predicted theoretically, owing to lack of knowledge of the properties of this concrete at the age at which these tests were made, the relations between the deflections of different slabs could easily be predicted.

In order to make a comparative study, all the deflections noted were reduced to terms of the deflections of a standard panel of a given depth. The assumptions made were as follows:

The deflection will vary directly as the load applied, directly as the cube of the span, inversely as the square of the effective depth, and inversely as the amount of reinforcement.

One of the test loads covered a panel only half as wide as it was long, and one of them, a short one, exhibited only one-fifth of the expected deflection. These two results are rejected in the following discussion. It was noted that the deflection from panels supported on two brick walls was considerably greater than that from those supported on two beams, although there seemed to be no difference between slabs having two beam supports and one wall support.

If it is assumed that the error arising from the use of the same denominator in the bending-moment formula is directly proportional in all cases to the variation in deflection, as exhibited in these standardized panels, the bending-moment formula should be as follows:

Supported at both ends, $\frac{w\,l^2}{8}$.

Both ends fixed, $\frac{w\,l^2}{14.3}$.

Mr. Noble.

If consideration is given to the fact that the slab supported on brick walls at both ends had a certain amount of negative reaction, the bending-moment formula will be as follows:

Supported at both ends, $\frac{w\,l^2}{9}$.

Fixed at both ends, $\frac{w\,l^2}{16.1}$.

In the writer's opinion, the latter assumption most nearly represents the facts, although, in his practice, he uses $\frac{w\,l^2}{12}$ for slabs supported at one end and fixed at one end, and $\frac{w\,l^2}{16}$ is only used where an ample amount of negative reinforcement is provided above the beam.

It is quite desirable that this matter be thoroughly investigated experimentally, as there is probably at no other point a greater opportunity for economy than in the proper study of negative moment reactions.

In discussing floor slabs rodded in both directions, it must be remembered that the ratio between the loads transferred to the longer beams and the shorter beams is not fixed by any mathematical formula, as is the case where the slab is a plate equally able to transfer stress in all directions. On the contrary, the distribution is quite largely under the control of the designer, and depends on the ratio between the longitudinal and transverse reinforcement.

It has been the writer's custom to assume that the total load transferred to the longitudinal beams is the same as that transferred to the transverse beams. The slab can be considered as divided into four parts by its diagonals. Each of these four triangles will be equal in area, and will represent the equivalent of the load transferred to the beam which forms its base. The center of gravity of the load, however, is one-sixth of the span from the supporting beam, while if a similar diagram is drawn for a slab rodded in one direction only, the center of gravity is one-fourth of the span away from the beam, and the load is twice as great. For this reason the writer considers that only one-third as much reinforcement in each direction is needed to secure equivalent carrying capacity, as compared with the slab rodded in one direction only.

In order to assure these conditions, the long direction of the slab must be the more heavily reinforced. As the ratio between the longitudinal and transverse directions increases, the reinforcement in the longitudinal direction increases very rapidly and soon eliminates any economy resulting from the consideration of change in position of center of gravity of the load. This is quite in distinction with the result of variation in ratio between the two sides of the panel, when

the rodding is done as suggested in the report of the Committee. In this case the allowable saving in reinforcement remains as great as ever, while the longitudinal bars in the extreme case must disappear. Mr. Noble.

In order to compare the proposed bending-moment formula with the result of the known tests, the writer has computed by this formula the safe load of various beams reported as being tested by A. N. Talbot, M. Am. Soc. C. E., in the now classic series of 1904, and this comparison is shown in Table 1. In making these comparisons only such beams as failed by tension in the steel are used.

TABLE 1.—COMPARISON OF BEAMS REPORTED IN BULLETIN No. 1, UNIVERSITY OF ILLINOIS, WITH PROPOSED FORMULA.

Formula: $M = f_s\ A\ j\ d.\quad j = \frac{7}{8}$.

Beam No.	Area of metal.	Kind of steel.	Safe M from formula, in foot-pounds.	Ultimate M actual test, in foot-pounds.	Factor of safety.
21	0.59	Mild.....	8 160	21 850	2.68
19	0.59	Mild.....	8 160	24 600	3.01
16	0.75	Mild	10 500	26 100	2.48
17	0.75	Mild.....	10 500	25 200	2.40
15	1.20	Mild.....	16 800	38 800	2.31
10	1.20	Mild.....	16 800	36 500	2.17
4	2.00	Mild.....	28 000	51 400	1.83
14	1.60	Mild.....	22 400	42 200	1.88
5	1.20	Mild.....	16 800	33 100	1.97
13	1.40	High carbon....	19 600	67 000	3.42
20	1.00	High carbon....	14 000	49 600	3.55
2	1.00	High carbon....	14 000	47 300	3.38
7	0.60	High carbon....	8 400	33 600	4.00
8	0.60	High carbon....	8 400	31 200	3.72

The average factor of safety of the nine beams reinforced with mild steel is 2.3; the greatest variation from this average is 31 per cent. The average factor of safety of the five beams reinforced with high-carbon steel is 3.61, and the greatest variation from the average is 11 per cent. The writer was much surprised to note that the use of high-carbon steel was evidently considered in the proposed specifications as of somewhat doubtful value. He was also surprised to note the low factor of safety, as shown by the formula, when compared with these tests. It would be interesting if the Committee would explain how these conclusions were reached. Surely the proposed specifications should be amended so that high-carbon steel can be used at a working stress somewhat near the same fraction of its elastic limit as is used for mild or medium steel.

The writer must confess to a great respect for re-rolled rails. This material is, everything considered, the cheapest reinforcing material on the market. At the time steel rails first began to be used for concrete reinforcement, the mills manufacturing this product did not fully understand the requirements of this trade. They were accus-

Mr. Noble. tomed, therefore, to roll rails of all sizes indiscriminately into reinforcing bars. The same shipment would contain material from rails of 90-lb. weight containing from 48 to 58% carbon, and rails of 50-lb. weight containing from 35 to 45% carbon. As a result, the product had a wide variation in elastic limit, and has come to be regarded among engineers as unreliable and unsatisfactory.

The writer has recently been using considerable quantities of rerolled material from a mill which makes a practice of selecting only the smaller rails for concrete reinforcement, and of testing carefully all the product. On a series of thirty-four tests, made on a recent shipment, the average elastic limit was 63 000 lb. per sq. in., the average ultimate strength, 109 300 lb. per sq. in., and the average elongation, 14.07 per cent. On a series of 142 tests of this material, taken just as they ran, the highest elastic limit found was 94 000 lb. and the lowest, 45 200 lb. The highest ultimate strength was 137 900 lb. and the lowest, 66 500 lb. The number of bars having an elastic limit below 5 000 lb. was only 8, and the number of bars having an elastic limit above 75 000 lb. was 10. Only 4 had an elongation below 10%, and 27 had an elongation above 20 per cent. Bending tests showed that the majority of these bars would bend around twice their own diameter provided the bend was made slowly, but if it was made on an anvil, by hammering into position, a small percentage would break before reaching this stage.

It is quite important that structural steel be ductile, as it is necessary to punch and shear it during fabrication, and also because of the vibration to which it is subjected. Neither of these reasons apply to reinforcing bars. They are not punched or sheared, are bent more economically hot than cold, and are not subject to any great amount of vibration. For this reason the writer prefers high elastic limit to ductility, particularly as high elastic limit is so productive of economy.

In the writer's opinion, the practice proposed by the Committee has two faults which, in a measure, compensate each other. In the first place, the factor of safety resulting from the proposed formula for moment of resistance is too low. In the second place, providing a sufficient negative reinforcement is used, the factor of safety resulting from the proposed bending-moment formula is too high. The result, therefore, is somewhat near the truth for standard conditions, but is apt to vary if applied rigidly to cases which differ from ordinary practice.

Mr. Russell. S. BENT RUSSELL, M. AM. SOC. C. E. (by letter).—After careful consideration of the reports made by the Committee, in the light of papers and discussions published previously in the *Transactions* of the Society, the writer is of the opinion that some of the points made by the Minority Report, in the second and third counts, are not without good foundation.

Mr. Russell.

On examination, the objection of the minority in its second count seems to be on the question of the scope of the inquiry. The report has gone beyond that field of construction which is subject to practical experiments, and has chosen certain forms of construction presumably in common use, and it may be thought, therefore, that the Committee has discriminated against other forms of construction.

In the writer's judgment, the report could be made stronger and more effective by limiting the field so as to confine the recommendations to matters which can be determined by practical tests. To this should be added, however, a plea for caution in applying the recommendations outside of their proper scope. In other words, the Committee should make recommendations for the design and for unit stresses to apply to rectangular and T-beams supported at the ends and for columns in axial compression. Sufficient experimental investigation has been made to warrant such recommendations, and they would always be subject to proof or correction by subsequent tests.

In the field of continuous beams and other complex structures, the Committee should avoid making definite recommendations; but, in order to prevent wrong uses of its specifications, it should certainly call attention to usual errors in the design of such structures, such as the omission of top reinforcement over the supports of continuous beams, lack of provision for eccentric loads, temperature stresses, etc.

The report could be made to conform to the lines herein suggested without much alteration in form, and it would then offer much less ground for criticism in reference to arbitrary recommendations.

It may be added that the proposed changes would also remove much of the objection shown in the third count of the minority, as the working stresses recommended would be in the field of practical tests.

Mr. Worcester.

J. R. WORCESTER, M. AM. SOC. C. E.—The reasons of the minority of the Committee for not signing the report of the majority are, in substance:

1.—That the report is hastily prepared, without careful consideration, and not in proper form.

2.—That it refers to one type of construction, without breadth enough to be applicable to designs which the future may develop.

3.—That the working stresses recommended are not supported by the evidence of tests.

4.—That the acceptance of the report would tend to encourage questionable practice.

The duties of a special committee of this Society are not definitely laid down, but are controlled by the judgment of the committee itself. This Committee was directed to "take up" the question of concrete and steel concrete. The reason for its appointment was the realization by the Society that there was grave danger in the rapidly increasing use of these materials without adequate scientific investigation and

Mr. Worcester. determination of properties, and the general diffusion of knowledge based on such studies. At the time, the reinforced concrete trade was controlled mainly by companies claiming proprietary rights and special knowledge of properties, and the public was dependent on the catalogues and contracting agents for information as to what dimensions and details to use. At the same time, many contractors, in absolute ignorance of the science, were claiming that they could handle the materials as well as any one. City authorities were being beset with applications for building permits based on material not mentioned in their laws, and were in grave perplexity as to what restrictions to impose. The cement manufacturers were as much troubled as consumers over the situation, realizing the danger to their trade from reckless use of their product. Under these circumstances, the Committee's understanding of its duties was to make sufficient study of the properties of concrete and reinforced concrete to enable it to lay down general rules for proportioning, which would result in safe construction. While the Committee has never formally adopted this wording as the objective at which it aimed, it seems to the speaker to have been so admitted, tacitly, by all members. Nearly five years elapsed from the time the Committee was appointed until this progress report was presented, and it can hardly be urged that undue haste was exhibited in giving something to the Society. To be sure, not all of the time was spent in the preparation of the report, but a full year was spent in this part of the work, and the remainder of the time in the accumulation of information. During the five years that the Committee has been in existence, many of the reasons for its appointment have disappeared, through the untiring efforts of many investigators and the gradual dissemination of knowledge. It is probable that if its report were delayed long enough, it might be saved the trouble of making any report at all, as it would not be needed. There still remains, however, a considerable divergence of views on many of the points treated in the report, as is evident from criticisms which have appeared. As to the form of the report not being correct, the majority would gladly see it improved, but considers form subordinate to substance.

In answer to the criticism of the minority that the report treats of one style of construction only, it may be said that it deals with every kind of stress to which concrete is subjected, and includes all ordinary conditions of proportioning and handling. If, in the future, new methods of designing introduce conditions not covered by these rules, they may be studied as developed. It is undoubtedly true that some special designs, involving indeterminacy of analysis by the ordinary rules of mechanics, are already extensively used, and these may well deserve experimental study. The existence of such designs, however, should not delay a report on other types which are in universal use and have been carefully investigated.

Mr. Worcester.

As to the tests on which the report is based, the majority of the Committee did not consider it necessary or useful to submit with its report the voluminous data at its disposal. Many of these data are already in print, and the remainder are likely to be published sooner or later. If they were all reprinted at this time, they would be of little use except to an investigator with unlimited time at his disposal, who wished to check the deductions made by the Committee. Moreover, the recommendations, particularly those contained in Articles III, IV, V, and VI, are largely the outgrowth of the experience in actual construction of members of the Committee and many others, rather than any definite series of tests. The majority feels safe in asserting that no recommendation is made in the report which is not based on good grounds. Any questions as to specific points in the report will be gladly welcomed by the Committee, and, in answering such questions, the majority may perhaps set at rest any doubts as to the sufficiency of evidence on which the recommendations are based.

The last charge of the minority, namely, that the acceptance of the report would tend to encourage questionable practice, is the most serious, and, if substantiated, would be sufficient reason for the refusal of the Society to give credence to the report. It is difficult to meet a sweeping assertion of this kind without specific mention of the particular features on which it is based. The fact that, since its presentation, this report has been freely criticized as being too conservative, is an indication that much of the practice of the day is more radical than that recommended. It may be, as asserted, that "the number of disasters in concrete structures has not diminished, but rather increased," but the minority cannot mean to imply that any disaster has ever occurred in construction executed in accordance with the recommendations of the report; and, it may be stated further, without fear of contradiction, that no failure has occurred in which principles laid down in the report have not been distinctly violated. In recommending unit stresses, the Committee has advised safety factors higher than those allowed in steel construction, and everywhere has correlated them to the quality of the concrete used, so as to insure the preservation of the desired factor. In the principles of design, no refined mathematical theories are advanced, but assumptions are adopted, which have been amply proved to be safe, and the Committee has repeatedly called attention to features wherein more experimentation may lead later to greater economy than can safely be recommended at the present time. The majority members of the Committee have felt their full responsibility to the Society and to the Profession, and have no interest in advocating any questionable practice. They rest assured that this charge will not be taken seriously by any engineer who makes a study of the report.

In conclusion, the speaker wishes to submit that, in view of the

Mr. Worcester. facts, the statements of the minority are not substantiated. Believing that concrete and reinforced concrete have become established among the most useful materials of construction, and are being used in every branch of engineering more extensively every day; that rules for the use of these materials have been adopted already by many nations, as well as municipal governments; that other technical societies are adopting rules based on no more complete data than are available to this Society; and that construction will not await absolutely complete scientific demonstration of a material before adopting it, are we not justified in advancing, in a progress report, tentative conclusions?

Mr. Douglas W. J. DOUGLAS, M. AM. SOC. C. E. (by letter).—The writer has read the Minority and Majority Reports of this Committee with much interest, and desires to offer some criticism. He is of the opinion that the report of the majority should be adopted, after some changes have been made in it and after additional matter has been inserted.

In regard to the Majority Report, the writer is of the opinion that it would have been strengthened very much if the data on which the permissible unit stresses were determined had been stated, and, while approving in a general way of the unit stresses adopted, he thinks that it would be a serious mistake to adopt the report without such additional information as would permit the Society to use the suggested unit stresses with a better understanding of the basis on which they were determined. One should bear in mind, in using the stresses recommended, that they are based on first-class work, and are high, unless the work is to be executed under more competent supervision than reinforced concrete work, in general, receives. Most building departments, improperly, do not require competent certificates of cement and steel tests, and are not equipped with competent inspectors, and such departments would not be justified in using the high stresses allowed for concrete in the Majority Report. The writer believes that the report should be referred back for revision, in order to perfect its usefulness, but he is strongly of the opinion that it is basically excellent, and needs only slight revision, in order that it may serve more fully the purposes for which it is intended.

In considering the Minority Report, it might be well to note that the art of reinforced concrete construction is severely criticized, such terms as, "infancy, pioneers, crude materials, and unscientific," being used, and by these expressions and the further statement that: "The number of disasters in concrete structures has not diminished, but rather increased; a word of warning, therefore, would be appropriate at this time," it is shown clearly that the minority is somewhat prejudiced against this type of construction.

In the Majority Report, it is stated:* "Plain concrete or massive concrete is well adapted for structural forms in which the principal

*Proceedings, Am. Soc. C. E., Vol. XXXV, No. 2, p. 88.

stresses are compressive." This includes "* * * in many cases, arches." It is thought that this latter statement is somewhat misleading, in that steel reinforcement in arches of ordinary rise and correct form is not economical, except where the probable deformation of the foundations makes reinforcement advisable. Mr. Douglas.

In Article II, Section 2, it is thought that Paragraphs 1, 2, 3, 5, and 7, would better be omitted, as they apply to all classes of structural design and not specifically to concrete work.

The writer thinks that it would be advisable to add the following to Paragraph 4 of this article: The design should show typical laps and bends and typical slabs, beams, girders, and columns, properly dimensioned, to insure the packing of all steel members. All beams which have been figured upon, in the design, as T-beams, should be indicated on the plans, and, if a sufficient number of shear bars have been incorporated in the design to take the entire horizontal shear between the stems and the flanges of the T-beams, this also should be noted on the design.

In regard to load tests, Article II, Section 3, it is believed that the test loads should not exceed twice the live load, as otherwise the elastic limit of the steel reinforcement may be exceeded.

The addition of the following is suggested to this paragraph: Tests should be made on unfinished floors (that is on floors upon which the finished surfaces of strip and strip filling have not been placed).

Tests should be made with materials, such as water or sand (not in bags), which do not arch themselves. Concentrated test loads should not be used except to determine the efficiency of the floor system to withstand concentrated loads.

In Article III, Section 2, it is stated:

"Mortars composed of one part Portland cement and three parts fine aggregate, by weight, when made into briquettes should show a tensile strength of at least 70% of the strength of 1:3 mortar of the same consistency made with the same cement and standard Ottawa sand."

This is an excellent requirement, and one which is seldom incorporated in specifications.

In Article III, Section 4, it is stated that the reinforcing steel should fulfill the requirements for structural steel. This is thought to be an unreasonable requirement, except where the reinforcing steel consists of built-up structural members. Such reinforcements, of course, would receive the same injurious treatment (such as punching) that structural steel members receive. Nor does it appear to be advantageous to exclude high-carbon steel, if such steel gives proper bending tests. There can be no reasonable doubt that, for reinforcing concrete, steel of a high elastic limit has material advantages over steel of a low elastic limit.

Mr. Douglas.

In specifying the quality of reinforcing steel which is to be bent, only, and then rarely, at angles of less than 90°, it should be sufficient to stipulate the minimum ultimate strength, elastic limit, and bending quality. Cold bending gives the best test to determine the suitability of steel reinforcement, and it is easily made in the field. The writer, therefore, suggests the following:

Reinforcing steel should have an ultimate strength of not less than 60 000 lb. per sq. in., and an elastic limit of at least 50% of its ultimate strength, and, for sizes $\frac{3}{4}$ in. or less in diameter, should bend cold 180° around its own diameter without fracture. For sizes greater than $\frac{3}{4}$ in. in diameter, it should bend 180° around twice its own diameter without fracture, except in cases where all bends necessary in the construction are to be made hot, in which case the steel should bend cold 90° around its own diameter without fracture. In the case of flat bars or structural shapes, the reinforcing steel should be bent cold around its own thickness, or twice its thickness, respectively. Tests on the ultimate strength, elastic limit, and bending quality of the steel should be made in the form (as to cross-section) in which it is to be used. Steel reinforcement made from rerolled rails should not be used. If deformed bars are to be used, only the net section—the section exclusive of all projections—of the areas of such bars should be considered effective.

Bars showing fractures at bends should not be used.

All reinforcing steel should be free from paint, dirt, oil, grease, scale, and rust flakes, before being encased in concrete.

It is thought to be impracticable and unnecessary to stipulate that steel should be free from rust, and the writer thinks that it would be better to substitute the term, rust flakes, instead of rust.

In Article IV, Section 3*a*, it is stated that special care should be exercised to prevent the formation of laitance. The writer knows of no way of doing this. The only practical way to prevent the formation of such a large quantity of laitance as will do material harm to the foundations is to lay the concrete as continuous work, and as quickly as possible.

In Article V, the Committee does not go very deeply into the question of forms, and as this is of vital moment, the elaboration of this article, to include the following requirements, is suggested:

All forms—and supports for forms—should be strong and stiff, true, out of wind, practically unyielding, and sufficiently tight to hold the liquid mortar without leakage; and their interior dimensions should conform to the dimensions of the concrete sections shown on the approved plans. The lagging—when lumber lagging is used—should be of a good quality of lumber, dressed to matched sizes. This lumber should be dressed on one face and two edges, except when the exposed surface is to be plastered, in which case the dressing should be omitted.

Mr. Douglas.

Forms which, at any time, become twisted or warped, should be removed or correctly adjusted before concrete is placed within them.

All forms—and supports for forms—should be designed to carry safely the weight of the forms, the liquid concrete being assumed to weigh 150 lb. per cu. ft., and a live load, in addition thereto, over the floor-slab area, of 25 lb. per sq. ft. They should also be designed to withstand safely a wind pressure of 20 lb. per sq. ft. upon all surfaces exposed to the wind.

The forms for beams, girders, and lintels should be designed so that at least one side of each beam and girder and lintel may be removed without disturbing the bottom portion of the forms and its supports. All posts supporting forms for slabs, beams, and girders should rest upon wedges which may be loosened and removed without producing undue strains in the floor system.

Posts which are to be supported by the ground should have their loads distributed by spread footings, so that there may be no appreciable settlement of the posts due to settlement of their foundations.

Columns with or without bands or hoops and with vertical reinforcement, in which there is a clear space of 5 in., or more, between the vertical units of rods, bars, or shapes, and where the vertical bars are wired so that the clear space between wires in the same horizontal plane is not less than 8 in., and where the vertical reinforcement has no attached web or shear members, should be poured—filled with concrete—for heights not exceeding 20 ft., without leaving one side of the column form open, except that, at the bottom of all columns, one side of each column form should remain open until about an hour before the filling of the column form is begun.

All columns not conforming to the above specifications should have one side of each column form open until concreting is begun in it, and its open side should be lagged up as the concreting progresses, but in no case should the top of this lagging be more than 3 ft. above the level of the column concrete.

In the case of columns of circular cross-section, the phrase, "one side open," should be taken to mean that, in horizontal cross-sections, at least one-sixth of the perimeter of the circular form should be open.

All shavings, chips, sawdust, and other foreign matter should be removed from within the forms before concrete is placed therein.

While agreeing with the report as to the time of the removal of forms, the writer thinks a general minimum might be stated as a guide, and suggests the following:

The season favorable to the laying of concrete (when the temperature does not fall lower than a few degrees below freezing) should be lengthened in the lower latitudes and shortened in the higher latitudes, and the unfavorable season should be correspondingly shortened or lengthened.

Mr. Douglas.

Forms should not be removed in less time than is specified below:

Slabs and lintel forms, April 1st, to November 1st:	
Spans less than 3 ft.	3 days.
More than 3 ft. and less than 5 ft.	4 days.
More than 5 ft. and less than 8 ft.	5 days.
More than 8 ft. and less than 13 ft.	6 days.
More than 13 ft.	7 days.
Slab and lintel forms, November 1st to April 1st.	12 days.
Supports for beams and girders, April 1st to November 1st.	15 days.
Supports for beams and girders, November 1st to April 1st.	21 days.
Columns and wall forms, April 1st to November 1st.	3 days.
Columns and wall forms, November 1st to April 1st.	12 days.

The above times of removal should be estimated from the time the concrete is completely placed in any section of the work.

Before undertaking to remove any form work, the superintendent or foreman in charge of the work should satisfy himself as to the condition of the concrete, and if it does not ring when struck by a hammer, he should call the attention of the engineer to the same and delay the work of removing the forms until the hammer test is satisfactory.

Before removing the temporary supports of the beams and girders, at least one side of the adjacent column forms and one side of each beam and girder form should be removed in order to expose the concrete to view, so as to give evidence of the soundness and hardness of the concrete.

When forms are being removed, there should be no load upon the portion of concrete affected in excess of one-sixth of the live load for which the portion affected was designed, unless temporary posts are left in to take care of such load.

Reinforced concrete floors—this does not apply to reinforced concrete slabs laid upon steel beams and girders—should be supported by continuous lines of temporary vertical posts—sufficient to carry the floor weights—extending down to the basement or to a floor which is strong enough to bear the upper floor loads. These props should be removed at such times as the upper floors have hardened sufficiently to carry safely their own load and such floors above them as are unable to carry their own weight safely.

The times of removal given hereinbefore are based on the assumption that the upper floors are sufficiently strong to carry their own weights, or that continuous lines of vertical posts have been provided to take care of the upper floor loads.

In Article VI, Section 1, it is stated that "in columns, large bars

should be properly butted and spliced." This paragraph also states that "in foundations, bearing plates should be provided for large bars or structural forms." It is thought that it should be specifically stated that the ends of large bars should be milled, and that, instead of requiring bearing plates, these requirements should read, bearing plates or bars. Wide plates are objectionable where columns rest upon spread reinforced concrete footings, on account of making a horizontal shearing plane of weakness. Mr. Douglas.

In Article VI, Section 2, it is stated:

"Girders should never be constructed over freshly formed columns without permitting a period of at least two hours to elapse, thus providing for settlement or shrinkage in the columns."

It is an open question whether or not it is necessary to pour a column in advance of the girder which it supports. The writer recently poured columns 42 ft. in length and immediately on their completion, the cross-girders were poured, and the work is entirely satisfactory. About the only condition which would make any difference would occur if the girder set up more rapidly than the column, or in the case of a very large column, as in a lower story of a building, supporting a small beam, in which case the column concrete in setting might rob the girder concrete to the detriment of the latter. And it further appears that if it were necessary to have a time interval between the pouring of the columns and the girders which they support, two hours would not be sufficient.

In Article VII, Section 1, it is stated that: "Massive concrete is also a suitable material for arches of moderate span where the conditions as to foundations are favorable." This sentence would conform more closely to the accepted practice if the words, "of moderate span," were stricken out.

In Article VII, Section 3*b*, it is thought that all beams and slabs estimated as continuous or restrained beams should have their span lengths taken from center to center of supports, and that the load upon the beams and slabs should be taken on the clear spans. In simple beams and slabs the span length should be taken from center to center of supports, but should not exceed the span length plus the necessary bearing length. The span length plus the depth may result in an absurd length of beam and of slab.

In Section 4, it is thought that a better shaped T-section would result if the additional restriction were made that the width of the flange should not exceed five times the width of the stem.

It is thought that the following additional restrictions might properly be added: If shear bars are necessary to take care of the shearing stresses, they should extend at least to the center of the slab, and there should be sufficient steel in the slab transverse to the

Mr. Douglas. beam or girder to take care of the negative bending moment in the slab, and said steel should be located so as to take care of the negative bending moment effectively. This restriction is important, because, if the negative bending moment is not taken care of, the beam may crack and reduce the effective width of the flange below that calculated in the design.

The writer would suggest the following at the end of Article VII, Section 6:

When the bottom reinforcing bars are bent up at the ends of a beam, for shear and negative bending moment, care should be taken that the bottom bends are not made so close to the middle of the beam that there is insufficient horizontal reinforcement near the supports to take care of the positive moment.

In Article VII, Section 7, it is stated in the last paragraph that:

"The lateral spacing of parallel bars should not be less than two and one-half diameters, center to center, nor should the distance from the side of the beam to the center of the nearest bar be less than two diameters. The clear spacing between two layers of bars should not be less than ½ in."

The writer is of the opinion that the allowed lateral spacing of bars, in systems which provide proper spacers or separators, may be less than in those which do not have such spacers or separators, because, in the latter systems, the horizontal bars become more closely bunched together than the drawings indicate. In many cases they are found in contact for a considerable portion of their lengths.

If there are two layers of bars, those of the upper layer should be spaced farther apart than those of the lower layer. If there is a third layer, the spaces between the bars of the uppermost layer should be still greater than those between the bars of the second layer.

It is thought that the introduction of the expression, "diagonal tension," in Article VII, Section 8, may be somewhat misleading to many, and, as the report of the Committee does not include any formula for diagonal tension, but only for shear, it would be better not only to omit this expression, but to state clearly that if the vertical and horizontal shear are taken care of properly, the diagonal tension need not be considered.

On page 104* general rules for shear reinforcement are given. If the Committee means that it is good practice to allow a value of 120 lb. per sq. in. for shearing stress in concrete, it would be of interest to the Society to know the experimental data on which this is based. These general rules might be better divided into:

a. For beams having horizontal bars only;

b. For beams in which the shear is partly taken care of by bent-up rods, shear bars, or both.

* *Proceedings*, Am. Soc. C. E., Vol. XXXV, No. 2.

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Under Section 8 it would be advisable to prohibit the eccentric attachment of shear bars to round rods, because this eccentric attachment tends to cause these bars to rotate, and thereby decreases the available adhesion.

In Article VII, Section 9, it is stated: "It is recommended that the ratio of the unsupported length of a column to its least width be limited to 15." The writer does not believe that the ratio of the unsupported length of the column to its least width should be limited to 15. This seems to be an unnecessary requirement to put on the designer who will occasionally need to support light loads, such as roof loads, by the use of long columns, and if he reduces the working stresses in the concrete and steel, there is no apparent reason why he should not use a column twenty-five times its least width, if this column is properly reinforced. It is suggested that, on the basis of the unit stresses allowed, the ratio should be limited to 15, and that if a higher ratio of length to least width is used, the allowable unit stress in the concrete steel be reduced on the basis of the following formula or one similar thereto:

$$F_c^1 = F_c^2 + 450 - \frac{30\,L}{D}.$$

F_c^1 being the unit stress (from 450 to 600) allowed in the column concrete for columns in which $\frac{L}{D}$ exceeds 15, and F_c^2 being the unit stress allowed in the column concrete for columns in which $\frac{L}{D}$ is 15 or less than 15.

Under Section 9, on columns, it would be advisable to state that if the structural steel portion of the column is in any form which tends to divide the column in its horizontal cross-section into independent parts, the concrete should be regarded as fire-proofing only. For instance, where the steel is in the form of an I or star shape, and in which the reinforcing steel extends to within a few inches of the outside of the concrete, it would not be good practice to assume that the concrete would carry any portion of the column load. By hooping the concrete it might be made to carry its portion of the load, but such design would be manifestly uneconomical.

On page 105,* working stresses in the concrete for various types of columns are recommended.

Is there any reason for allowing as high stresses in the concrete of a structural reinforced column as in that of a hooped column? One cannot regard a column built of structural shapes as a hooped column. Hooping must be circular, not square. Bands built in the form of squares would certainly strengthen the column, but it is believed to a much less degree than if the bands were circular.

* *Proceedings*, Am. Soc. C. E., Vol. XXXV, No. 2.

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It is thought that the allowable unit stresses in *d* should be intermediate between those of *a* and *b*.

In connection with the working stresses in columns, the writer thinks that in *a* and *d*, a limit to the percentage of steel reinforcement should be stated. A high percentage of steel, say 9% or 10%, would cut up the column concrete so that it would not be safe to stress it as highly as if a lower percentage of steel had been used.

The writer suggests the consideration of a specification in general in accordance with the following:

For the sake of stating a general rule, 600 lb. per sq. in. is assumed as good practice for 5% reinforcement.

In columns with longitudinal reinforcement only, the following unit compression per square inch will be allowed:

For columns not exceeding 5%, reinforcement 600 lb.
For columns exceeding 5% but not exceeding 6%, 575 lb.
For columns exceeding 6% but not exceeding 7%, 550 lb.
For columns exceeding 7% but not exceeding 8%, 525 lb.
For columns exceeding 8% but not exceeding 10%, 500 lb.

Article VIII, Section 2, states:

"When compression is applied to a surface of concrete larger than the loaded area, a stress of 32.5% of the compressive strength at 28 days, or 650 lb. per sq. in. on the above-described concrete, may be allowed. This pressure is probably unnecessarily low when the ratio of the stressed area to the whole area of the concrete is much below unity, but is recommended for general use rather than a variable unit based upon this ratio."

It appears that the allowable compression for the load area is stated too indefinitely, and that the pressure allowed is low.

The rule which specifies the allowable unit pressure "when compression is applied to a surface of concrete larger than the loaded area," gives the designer too much leeway. For instance, he might take advantage of the higher allowable compression, when the surface of the concrete was only 5 or 10% greater than the loaded area. On the other hand, if the area of the concrete surface was ten times the loaded area, the allowable unit stress is low.

The writer would suggest the following, which he believes is based on tests, and on sound logic, independent of recorded tests:

When a compression of three times the loaded area and less than ten times the loaded area, is applied to a surface of concrete, a compressive stress of 800 lb. per sq. in. of the loaded area will be permitted. When a compression of more than ten times the loaded area is applied to a surface of concrete, a compressive stress of 1 000 lb. per sq. in. of the loaded area will be permitted.

In Article VIII, Section 6, it is noted that the adhesion allowed for drawn wire is less than that allowed for bars. The reason for this

is not apparent, because, while it is of uniform cross-section, it is far from straight. Mr. Douglas.

The writer would suggest the desirability of publishing, as part of this report, the correct method for applying the shear formula given on page 118,* as the average engineer engaged in reinforced concrete work finds the application of this formula somewhat difficult.

For simplification, the writer advocates that the shear in restrained and continuous beams be assumed as equal to that of simple beams.

The following additional data are believed to be worthy of consideration in the formulation of general rules for the design and construction of reinforced concrete work, and might properly be considered by the Committee, should the report be referred back for revision:

After the concreting of a slab, beam, or girder is begun, it should be carried on continuously until the full depth of the slab, beam, or girder is reached. By "continuously" is meant that adjacent layers of concrete should be placed within 90 min. of each other.

When T-beams or T-girders have been figured on in the design—which should be indicated on the plans—the slab should be laid continuously with the beam or girder concrete.

Beams and girders which are calculated as simple beams, but are built continuously over supports, should be reinforced at the supports, in the upper portion of the beam or girder, with steel having a cross-section equal to one-quarter of the cross-section of the steel necessary in the bottom of the beams and girders at their middle. This upper reinforcement is introduced in order to prevent cracking, and should extend a sufficient distance beyond the center of the supports in order to develop adhesion equal to the strength of the upper bars, on the basis of the allowable unit stresses given.

Reinforcing steel for slabs, calculated to carry stresses, should not be spaced farther apart than two and one-half times the depth of the slabs, for slabs not exceeding 4 in. in depth, and not farther than twice the depth of the slab, for slabs exceeding 4 in. in depth.

Main tension members, for slabs, beams, and girders, should not be spliced except over supports.

The depth of beams and girders—measured below the bottom of the slabs, in the case of T-beams—should not exceed three times the width of the beam.

The steel of beams and girders should be designed so as to tie the building together securely at story levels.

Notches in the walls to receive the bearing ends of slabs should not be permitted, unless the height of the notches be three or more times their depth—horizontal dimensions being measured at right angles to the wall. Where notches are used at the bearing ends of slabs, the

* *Proceedings*, Am. Soc. C. E., Vol. XXXV, No. 2.

Mr. Douglas. reinforcing steel should extend into the notches to at least one-half their depth.

Floor slabs designed to carry heavy concentrated loads should be provided with transverse as well as longitudinal reinforcement, so as to distribute the concentrated loads.

In combination concrete and tile construction, the dead and entire live loads should be assumed to be carried by the concrete beams.

At floor levels, where all the column reinforcing rods are designed to abut, additional steel rods should be used to tie the columns in consecutive stories. This steel should have an area of at least two-thirds of 1% of the effective area of the column, and should be of such length that the adhesion will develop the strength of the bonding steel.

Longitudinal steel column reinforcement should not be butted or spliced farther than 2 ft. from floor levels.

Where steel reinforcement is not required in a column for compression, an amount of steel at least equal to two-thirds of 1% of the cross-sectional area of the column should be used.

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PAPERS AND DISCUSSIONS

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FIRE-RESISTANT CONSTRUCTION OF BUILDINGS.

An Informal Discussion at the Annual Convention, July 8th, 1909.*

By Messrs. Herbert M. Wilson, R. W. Lesley, Richard L. Humphrey, and S. Whinery.

Herbert M. Wilson, M. Am. Soc. C. E. (by letter).—The inventory of natural resources prepared by President Roosevelt's Conservation Commission has opened new lines of thought and investigation to the Engineering Profession as well as to other departments of the social fabric. Mr. Wilson.

Architects and engineers have realized in a general way the waste of property values involved in building and engineering construction not designed to resist such destructive agencies as fire and earthquake. A cursory examination, however, of the statistical results presented to the Mineral Section of the Conservation Commission shows the character of the waste by differentiation into its components of lumber, masonry, steel, and miscellaneous building material.

As many of the members of this Society well know, the Conservation Commission was divided into four sections, charged with the study of the waste of natural resources, as follows: (1) Water; (2) Forests; (3) Land, and (4) Minerals. The Federal Commission was fortunate in having at its disposal the accumulation of more than a quarter of a century of research by Government Bureaus charged with the investigations of the problems under consideration, foremost among which were the Geological Survey, the Forest Service, and the General Land Office.

The Secretary of the Mineral Section of the Conservation Commission is Dr. Joseph A. Holmes, Expert in Charge of the Technologic Branch of the Geological Survey, and to the writer, as Chief Engineer

* The discussion of this subject, for which no formal paper was presented, is printed in *Proceedings* in order that the views expressed may be brought before all members of the Society for further discussion.

Mr. Wilson. of that branch, was assigned the inquiry into the waste of mineral resources involved in building and engineering construction. He is indebted to Dr. Holmes and to the Director of the Geological Survey, Dr. George Otis Smith, for permission to present to this Society some of the results of this investigation.

The Supervising Architect of the Treasury Department has under his control public buildings costing more than $200 000 000, and is spending each year more than $20 000 000 in the construction of new buildings. At his behest, in order that he may design these buildings so as to render them most resistant to fire and earthquake at a minimum expenditure, inquiries are now being conducted in the field by economic geologists skilled in the study of the building stones, gravels, sands, and clay deposits of the country which are most suitable and most cheaply accessible to the three hundred cities scattered throughout the United States in which public buildings are now under construction or projected. The samples are transmitted to the Pittsburg Laboratory of the Geological Survey for analysis and test as to their qualities either as aggregates for concrete or building stone or brick.

In co-operation with the Underwriters' Laboratory at Chicago, and independently at the Government Testing Laboratory at Pittsburg, investigations are being conducted into the rates of conductivity, and the fire-proofing and fire-resistant properties of various structural materials, including strength under load before and after subjecting to fire. Tests of the crushing strength of large samples of building stones, of beams, columns, and floor-slabs of concrete and reinforced concrete, and of building brick and tile, are likewise being conducted in order to determine which will give the most efficient results in building construction of the highest type. This feature of the inquiry has in view the fact that the Government should maintain as high a standard of efficiency in the construction of public buildings as is commensurate with economic design, since such buildings are intended to be of a permanent and enduring nature. The magnitude and importance of this work are such that the Government cannot afford to take any risk concerning methods of construction or materials to be used. The fact that the Government does not insure its buildings against loss by fire makes it especially necessary that it be provided against such loss by making them fire-proof. It is estimated that if the public buildings of the United States were insured, the cost to the Government would be more than $600 000 annually.

A startling fact developed in considering the value and fire risks of Government buildings is that the total cost of fires in 1907 amounted to almost one-half the cost of new building construction in the country for the year. The cost of building construction in forty-nine leading cities of the United States reporting a population of less than 18 000 000, amounted to $661 021 286, and the cost of building construc-

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tion for the entire country in the same year is conservatively estimated at more than $1 000 000 000. A preliminary inquiry had shown that the cost of fires to the people was nearly $500 000 000, for the year 1907.

The preliminary estimate that nearly one-half the value of all new buildings constructed in the United States in one year was destroyed by fire, was sufficient warrant for a thorough statistical inquiry, which would develop the facts for the whole United States, in connection with the specific inquiry for the Government regarding fire-proof and earthquake-resistant construction. This inquiry included the value of the property destroyed by fire, the maintenance of fire departments, the payment of insurance premiums less the benefits returned, protective agencies, and the additional cost of city water supplies, etc. It has developed the fact that the total cost of fires in the country in 1907 amounted to more than $456 485 000, or about $1 250 000 daily. This cost was five times as much per capita as in any country of Europe, and was equivalent to a tax on the people exceeding the total value of the gold, silver, copper, and petroleum production. This fire cost was greater than the true value of the real property and improvements in any one of the following States: Maine, West Virginia, North Carolina, North Dakota, South Dakota, Alabama, Louisiana, or Montana. This means that the total destruction of the real property in any one of these States in a year would not represent a loss greater than that caused by fires in the whole United States.

The actual fire losses due to the destruction of buildings and their contents amounted to $215 084 709, or a per capita loss for the United States of $2.51.

These actual fire losses are greater than the true value of the real property in the States of Utah, Delaware, Florida, Idaho, Wyoming, or Nevada. If a storm or earthquake devastated any one of these States completely, there would be a great outcry. Assistance would be rushed to the stricken people; a mighty attempt would be made to find a remedy for the conditions, and the activity would not cease until the remedy had been found. But, with the fires scattered throughout the country and the loss distributed among hundreds of cities and towns, there is no appreciation of the seriousness of the conditions, and very little attempt to better them. In many cities a big fire loss for the year is looked upon complacently, as if it were a natural and unavoidable condition. The significance of these figures lies in the fact that the loss in the cities of the six leading nations of Europe amounted to 33 cents per capita, or about one-eighth of the above. In addition to this waste in wealth and natural resources, 1 449 persons were killed and 5 654 injured in fires. As convincingly set forth, at the recent conservation meeting of the Engineering Societies in New York City, by Mr. Charles Whiting Baker, the buildings consumed, in the year, if placed on lots of 65-ft. frontage, would have lined both sides of a

Mr. Wilson. street extending from New York to Chicago. A person journeying along this street of desolation would have passed in every thousand feet a ruin from which an injured person was taken; at every three-quarters of a mile in this journey he would have encountered the charred remains of a human being; and, when burned out, at the end of a year, the fire would begin all over again on an even longer street.

An analysis of these fires losses, with a view to making the results of the structural materials investigations useful, not only to the Government, as the greatest consumer of such materials, but also to the various States and municipalities, and to the people of the country as a whole, showed that it would be desirable to point out, not only the most efficient methods of construction for each locality, but also how such improved construction might be brought within the cost limits of all the people. In this analysis, the fact stands out prominently that much of the fire waste is due to fires extending beyond the limits of the buildings in which they started. Exact figures as to the losses due to exposure were not obtainable, but the most conservative estimate indicates that at least 27% of the losses resulted from fires extending beyond the building of origin. Thus nearly one-third of these losses are due to the inflammable construction of American buildings, for in Europe, where fire-resistant construction prevails and building regulations are designed and enforced with a view to restricting the progress of a fire, there is practically no loss from exposure fires. And it is even more significant that only $68 000 000 of the loss from fires in the United States was in brick, concrete, and other slow-burning construction, while double that amount, or $148 000 000, was in frame buildings.

The investigation into the actual cost of fires in the United States was quite thorough, and included a complete inquiry into the total cost of water-works systems and the part of the construction and equipment, the annual expense, depreciation, taxes, interest, and maintenance costs chargeable to fire service; also the total cost of fire departments, the annual expense, depreciation, taxes, interest, and maintenance charges. The results showed $145 604 362 as the amount of fire premiums paid to the insurance companies above the losses paid by them; $28 856 235 as the total annual expense of the water-works upkeep chargeable to fire service; $48 940 845 as the total annual expense of fire departments; and $18 000 000 as the total annual cost of fire protection.

The results of this general inquiry into the causes of fires are summarized in Table 1.

A careful examination of Table 1 tells the story to the engineer. In studying it, he should bear in mind that, in the last thirty years, the total fire waste, being only the value of property destroyed, amounted to $4 484 000 000.

TABLE 1.—CAPITAL INVESTED FOR FIRE PROTECTION, AND ANNUAL LOSS AND EXPENSE ON ACCOUNT OF FIRE, IN THE UNITED STATES IN 1907.

		Investment in construction and equipment.	Annual loss and expense.
FIRE LOSS:			
Total fire loss	$215 084 709	.....	$215 084 709
FIRE PROTECTION: INSURANCE:			
Amount of fire premiums paid above amount of losses paid	(a) 145 604 362	.....	145 604 362
COST OF WATER-WORKS, CHARGEABLE TO FIRE SERVICE:			
Total cost water-works (construction and equipment) chargeable to fire service	(b) 245 671 676	$245 671 676	.....
Source of water supply (construction and equipment) chargeable to fire service	$66 482 220	.....	.....
Distributing system (construction and equipment) chargeable to fire service:			
2 016 927 tons of metal	127 236 668	.....	.....
350 159 hydrants	29 761 400	.....	.....
Separate high-pressure fire service	22 191 388	.....	.....
Total annual expense of water-works, chargeable to fire service		28 856 235	28 856 235
Depreciation and taxes, water-works, chargeable to fire service	10 563 881	.....	.....
Interest charge, water-works, chargeable to fire service	(c) 9 826 867	.....	.....
Maintenance, water-works, chargeable to fire service	8 465 487	.....	.....
FIRE DEPARTMENTS:			
Total cost of fire departments (construction and equipment)		107 063 594	48 940 845
Total annual expense of fire departments	48 940 845	.....	.....
Depreciation and taxes—fire departments	4 603 731	.....	.....
Interest charge—fire departments	(d) 4 282 540	.....	.....
Maintenance—fire departments	40 054 574	.....	.....
PRIVATE FIRE PROTECTION:			
Total cost, construction and equipment, private fire extinguishers, automatic sprinklers, etc	50 000 000	50 000 000	.....
Total annual private fire protection	.....	.....	18 000 000
Total fire waste		$402 735 200	$456 486 151

(a). The amount paid by insurance companies on account of fire loss was $114 164 469, and the amount received by them in premiums was $259 768 831.
(b). This is 22½ of the total cost of water systems, domestic and fire service combined.
(c). $245 671 676, cost of water-works, chargeable to fire service, capitalized at 4% interest, is equal to an annual charge of $9 826 867.
(d). $107 063 594, cost of fire departments, capitalized at 4% interest, is equal to an annual charge of $4 282 540.

Mr. Wilson. The general inquiry included not only the cost of fires in the United States, but also the cost in Europe, including fire waste, excess of premiums over insurance paid, annual expense of city water-works and fire departments chargeable to fire service, and private fire protection in a number of countries. From the results obtained, it is evident that the United States is paying annually a preventable tax of more than $366 000 000, or nearly enough to build a Panama Canal each year. The estimate gives the total cost of fires in the United States, if buildings were as nearly fire-proof as they are in Europe, as $90 000 000.

The figures are set forth in Table 2.

TABLE 2.—ANNUAL LOSS AND OUTLAY IN THE UNITED STATES, ON ACCOUNT OF FIRES; WITH COMPARISON SHOWING PROBABLE ANNUAL LOSS AND EXPENSE, IF BUILDINGS WERE AS NEARLY FIRE-PROOF AS IN EUROPE.

	United States, 1907.	Per capita.	United States, if buildings were as nearly fire-proof as in Europe.	Per capita.
Total loss by fire	$215 084 709	.....	$41 000 000	.....
Excess of premiums over insurance paid	145 604 362	.....	28 000 000	.....
Annual expense of water-works, chargeable to fire service	28 856 235	.....	6 000 000	.....
Annual expense of fire departments	48 940 845	.....	10 000 000	.....
" " " private fire protection	18 000 000	.....	5 000 000	.....
Total fire waste	$456 486 151	$5.34	$90 000 000	$1.05
Total loss by fire	$215 084 709	$2.54	$41 000 000	$0.48
Annual expense of fire protection	241 401 442	2.82	49 000 000	0.57

In Table 2, and more particularly in the following discussion, it should be borne in mind that, while fires will continue to occur in buildings and destroy them and their contents to a greater or less extent, the fire which extends beyond the building of origin and to its neighbor, and thereby produces a conflagration, does so only because the building construction is of the most inflammable nature. The costs which are here recorded, and exceed in amount those due to fire waste, are preventable only in so far as they are an incident to conflagration conditions as distinguished from fires restricted to the building of origin. The statements set forth relative to the possible reductions in the cost of fire departments, city water supply, and private fire protection charges, must be considered only in connection with those service charges which are due to combatting ever-present conflagration conditions.

The insurance and property loss in the destruction of the City of San Francisco by fire and earthquake in 1906 amounted to about

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$315 000 000. The area burned was 3 000 acres, embracing 520 city blocks, on which 25 000 buildings were destroyed, and it is notable that of these only 3 000 were of brick or stone, and 22 000 were of frame construction.

Since 1866, the conflagrations of the United States have cost, in fire waste alone, $936 551 135. This loss, according to European standards, is needless, and is preventable. If, as it has been shown, the actual cost of fire protection in all its phases is more than the fire waste, it is fair to assume that more than $1 000 000 000 has been expended in this period in an attempt to take care of these conflagrations. This, also, is an unnecessary expenditure, if European standards are considered, and makes a total tax from conflagrations alone since 1866 of more than $2 000 000 000. This is true because the conflagration loss in any of the European countries is almost nothing. It is indeed rare when a fire extends beyond the building in which it starts.

Notable conflagrations in the United States have been those of Chicago, in 1871; Boston, in 1872; Baltimore, in 1904; and San Francisco, in 1906. The year 1908 has a conflagration record exceeding that of 1907, one such disaster alone, that of Chelsea, Mass., involving an insurance loss, not taking into account the incidental losses, of $8 846 879.

The analysis of the fire losses in the United States in 1907 is given in Table 3.

TABLE 3.—FIRE LOSSES IN THE UNITED STATES FOR 1907.

Statistics Gathered by the United States Geological Survey.

	Total.	Urban.	Rural.
Total fire loss	$215 084 709	$107 093 283	$107 991 426
Fire loss: buildings	109 156 894	50 173 625	58 983 269
Fire loss: contents	105 927 815	56 919 658	49 008 157
Total, brick, etc., fire loss	68 425 267	48 908 744	19 516 523
Brick, etc.: fire loss, buildings	31 092 687	19 816 474	11 276 213
Brick, etc.: fire loss, contents	37 332 580	29 092 270	8 240 310
Total frame fire loss	146 659 442	58 184 539	88 474 903
Frame fire loss: buildings	78 064 207	30 357 151	47 707 056
Frame fire loss: contents	68 595 235	27 827 388	40 767 847
Total number of fires	165 257	105 406	59 851
Number of fires in brick, etc., buildings	36 140	25 297	10 843
Number of fires in frame buildings	129 117	80 109	49 008
Loss per capita	2.51	2.54	2.49

From Table 3 it appears that the loss is rather evenly divided between urban and rural population. The large losses in cities and villages are not surprising, for not only do they contain many inflammable buildings filled with valuable property, but their proximity renders them particularly subject to exposure risks. The equally large

Mr. Wilson. losses in rural districts, where buildings are widely separated, can only be explained by the assumption that the remarkable efficiency of the city fire departments prevents even greater losses in urban fires, and that the buildings and contents in the rural districts are generally totally destroyed when fire breaks out. This is indicated in the total of the building losses, which amounts to more than $50 000 000 in cities, while in rural districts the loss in buildings, as distinguished from contents, is nearly $60 000 000. The contents loss in cities amounts to nearly $57 000 000, as against $49 000 000 in rural districts, though the contents of city buildings amount in value to several times those in rural districts. The much heavier losses in masonry buildings in cities as compared with those in the country are due to the rarity of buildings of this character in rural districts. The losses in frame buildings in cities and villages, amounting to more than $30 000 000, as compared with losses in buildings of the same character in the country, amounting to nearly $48 000 000, can be attributed to the efficiency of the fire departments and the utter lack of such protection in rural districts. In European cities, the construction of frame buildings is absolutely prohibited in all municipalities, and few are erected in rural districts, owing to the relatively high price of timber, In the United States, the conditions are the reverse. Until recently timber has been the cheaper, as well as the more easily worked, material, as compared with brick, stone, and steel. More than two-thirds of the total loss in the United States in 1907 was due to frame buildings. The exact losses were: $146 695 442 in frame buildings, and $68 425 267 in masonry and steel buildings.

As shown in Table 4, in those States in which the timber supply is abundant, there is an increased per capita loss of 59 cents over the per capita loss in the comparatively treeless States.

TABLE 4.—THE PER CAPITA FIRE LOSS FOR 1907 IN ELEVEN STATES WHERE TIMBER IS SCARCE, AND IN ELEVEN STATES WHERE TIMBER IS PLENTIFUL.

Statistics Gathered by U. S. Geological Survey.

	Total population.	Total fire loss.	Per capita.
Group 1. States having scarcity of timbers: Iowa, Ill., Okla., Conn., Del., N. J., S. Dak., R. I., Kans., Nebr., and N. Dak	16 785 460	$38 606 558	$2.30
Group 2, Timbered States: Wash., La., Tex., Miss., Wis., Ark., Mich., Pa., Minn., Ore., and N. C...	25 569 533	73 895 950	2.89

Studying these fire losses by geographical divisions of States—a division generally used by the Census Bureau—as set forth in Table 5, a remarkable feature is the large per capita loss in the Southern

States, namely, \$3.66, or more than \$1.00 in excess of the per capita loss in any other division. The cause lies in the fact that the Southern States are well timbered, and, in addition, suffer from the handicap of inefficient fire protection in the cities and villages. Mr. Wilson.

TABLE 5.—THE PER CAPITA FIRE LOSSES FOR 1907 IN THE UNITED STATES, AS SHOWN BY GEOGRAPHICAL DIVISIONS OF THE STATES.

Statistics Gathered by U. S. Geological Survey.

Geographical Division.	Total population.	Total fire loss.	Fire loss per capita.
North Atlantic: Me., N. H., Vt., Mass., R. I., Conn., N. Y., N. J., and Pa.	23 779 013	\$59 447 532	\$2.50
South Atlantic: Del., Md., D. C., Va., W. Va., N. C., S. C., Ga., and Fla.	11 574 988	25 349 223	2.19
North Central: Ohio, Ind., Ill., Mich., Wis., Minn., Iowa, Mo., N. Dak., S. Dak., Nebr., and Kans.	29 026 645	68 793 148	2.37
South Central: Ky., Tenn., Ala., Miss., La., Tex., Okla., and Ark.	16 368 558	59 908 922	3.66
Western: Mont., Wyo., Colo., N. Mex., Ariz., Utah, Nev., Wash., Ore., and Cal.	4 783 557	12 676 426	2.65

Statistics gathered by United States consular officers in Europe, at the instance of the National Board of Fire Underwriters, showing fire losses in six of the most prominent of the European countries, give the surprisingly low per capita loss of 33 cents. Had the United States such a per capita loss, instead of one of \$2.51 for a total estimated population for 1907 of 85 532 761, the total fire waste would have amounted to only \$28 623 290, or a saving of natural resources from fire, alone, to the extent of \$186 461 419.

These facts are set forth in Table 6, which shows that the average loss per capita is \$0.33, as compared with \$2.51 in the United States.

TABLE 6.—FIRE LOSSES IN SIX EUROPEAN COUNTRIES.

Statistics Gathered by The National Board of Fire Underwriters.

Country.	Years.	Annual average fire loss.	Population, 1901.	Loss per capita.
Austria	1898–1902	\$7 601 389	26 150 597	\$0.29
Denmark	1901	660 924	2 588 919	0.26
France	1900–1904	11 699 275	38 595 500	0.30
Germany	1902	27 655 600	56 367 178	0.49
Italy	1901–1904	4 112 725	32 449 754	0.12
Switzerland	1901–1905	999 364	3 325 023	0.30

Table 7 is a comparison of America's needless waste and Europe's prudence by grouping the per capita losses in cities of approximately the same size in the two countries.

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TABLE 7.—FIRE LOSSES IN AMERICAN AND EUROPEAN CITIES OF THE SAME SIZE.

Statistics Gathered by U. S. Geological Survey and Bureau of Manufactures.

EUROPEAN LOSSES FOR 1904.				UNITED STATES LOSSES FOR 1907.			
City.	Population.	Fire loss.	Per capita.	City.	Population.	Fire loss.	Per capita.
Paris, France	2 714 068	$1 266 282	$0.47	Chicago, Ill	2 049 185	$3 987 105	$1.43
Frankfort, Germany	324 500	99 492	0.31	Cincinnati, Ohio	345 290	1 971 217	5.70
St. Petersburg, Russia	1 500 000	2 198 541	1.42	Philadelphia, Pa	1 441 735	2 089 522	1.45
Birmingham, England	550 000	226 506	0.41	Baltimore, Md	553 669	916 603	1.66
Sheffield, England	426 686	75 989	0.18	Cleveland, Ohio	460 000	515 194	1.12
Toulon, France	101 602	55 391	0.55	Atlanta, Ga	104 984	225 237	2.15
Bremen, Germany	208 847	78 372	0.38	St. Paul, Minn	204 000	522 447	2.56
Molenbeek, Belgium	63 678	106 150	1.67	Evansville, Ind	68 957	196 702	3.08
Laiken, Belgium	31 121	22 349	0.72	Oshkosh, Wis	31 033	80 500	2.59
Etterbeek, Belgium	28 992	19 504	0.81	Easton, Pa	25 238	32 073	1.27

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Realizing that the loss from fires due to the destruction of property involves only a fraction of the total loss, a careful statistical inquiry was made into the additional cost of water supplies necessary for combating conflagrations. This inquiry developed the fact that in 1907 the water-works systems of the United States were valued at $1 129 247 532, the cost chargeable to domestic service being accepted as $883 575 856, and that chargeable to fire service $245 671 676, or 22% of the whole.

In these systems the pipe used amounted to 7 097 800 tons, of which 5 080 873 tons were used for domestic service and 2 016 927 tons for fire service. The total number of hydrants was 420 394, and the number of hydrants in fire service, 350 152. The cost of hydrant fire service was $29 761 400, and the value of established or contemplated high-pressure fire systems amounted to $22 191 388.

The facts covering the property values involved in additional water supplies for combating fires were secured after much difficulty, and, as any engineer must know, are neither as complete nor as satisfactory as could be desired. To engineers and superintendents of water-works, 5 700 report blanks were mailed and an extensive personal correspondence was taken up in order to secure data which were not furnished on the first call in such condition as would make them useful in the investigation. Only a small percentage of the replies were complete enough for use in tabulation. Under these circumstances, recourse was had to the total cost of water supplies, as secured by the census for cities having a population of 30 000 and more and owning water-works. Data for cities having a population of less than 30 000 were obtained from the "Spectator's Year-Book." These data were then segregated to show the total cost under five geographical divisions of the United States, and, finally, the total cost in each of these geographical divisions was subdivided in order to show the total cost in cities of four classes of population, as follows: Cities of less than 5 000; between 5 000 and 30 000; between 30 000 and 100 000; and with more than 100 000 inhabitants.

By adopting the same plan of classification in tabulating the reports received from engineers and superintendents of water-works, who gave detailed information, it became possible to apply percentages to the total cost of water-works systems for the corresponding geographic divisions and classifications by sizes of cities, and, from these, the results given were obtained. The geographical divisions are arranged in accordance with the different requirements in the various localities, and are the result of personal inspection and interviews with hydraulic engineers and superintendents of water-works in a number of typical cities throughout the United States. In the prosecution of this inquiry the writer received much valuable advice and assistance from many members of this Society throughout the United States, several

Mr. Wilson. of whom spent long hours in going over their plans for distribution in the cities under their charge or with the operation or design of which they were connected. In this manner, only, has it been possible to arrive at any reasonable approximation as to the additional cost of water supply and water distribution made necessary by having a reserve for fire fighting under conflagration conditions.

This inquiry was not based on the idea that water-works are considered for two purposes of equal importance, viz., domestic water service and fire-protection service, but on what it would cost for water-works systems for fire protection alone. The governing idea has been to ascertain, in any city having a modern water-works system, what it would have cost, in the matter of storage or pumping capacity, in the extent and dimensions of the distributing system, and in the number of hydrants, should the city design them anew with a view to supplying only domestic needs and such minor service as would permit fire fighting, with a view to confining the fire to the building of origin, with the latter built of fire-resistant materials. In the New England and Middle States much consideration has been given to the subject of increasing water supplies which were not originally planned to meet the enormous growth in population which has taken place in the last two decades. Pumping service and distributing systems in the hearts of such cities have had to be entirely replaced with larger primary supplies, and in some have had to be duplicated. In a few cities a separate high-pressure fire service has been installed. On the other hand, throughout the Southern States, this feature of increasing existing supplies has been given little consideration. In the more modern cities of the Central States, the city service was originally designed in the light of the most modern needs and practice. In the Far Western States, especially in the small places, the requirements of domestic supply, on account of irrigation of gardens and city yards, preponderates to such an extent that practically none of the water supply has been considered as of account for fire protection, the domestic supply being of such magnitude as to take care, not only of ordinary fires, but also of conflagrations. This is less true in cities with populations of more than 30 000. In the humid regions the domestic requirements are so great in cities of 250 000 or more inhabitants, that but a small percentage of the water supply need be considered as of account for fire protection. For these reasons the subdivision was made by population as well as by geographical distribution.

Much careful consideration has been given these matters from time to time in the past especially by J. T. Fanning, M. Am. Soc. C. E. In a paper on "Distribution Mains and Fire Service" before the American Water Works Association in 1892, he considers that domestic consumption for all purposes may be treated uniformly as 66⅔ gal. per capita per day, as compared with the very much greater per capita supply

furnished in the large cities on account of fire service and domestic consumption. For a city of average population, say 100 000, he adopted a rate of flow for fire service of 10 000 000 gal. per day, and for combined domestic and fire service, 16 666 666 gal., or about 60% of the water supply on account of fire service. In a later paper, before the same Association, at its Twenty-sixth Annual Convention, Mr. Fanning furnished some additional information on this subject, in which he found that the portion of the main required for the domestic flow at the times when there is also a fire flow, and for different populations, was as follows: for 5 000 population, 11.24%; for 10 000 population, 19.52%; for 25 000 population, 43.75%; for 50 000 population, 57.14%; and for 100 000 population, 88.89 per cent. Mr. Wilson.

The outcome of the investigation by the Geological Survey indicates that, on the average, for the whole United States, 22% of the total expenditure on behalf of public water supplies, or less than one-fourth of the total cost of water-works systems, is due to the additional supplies necessary for protection against fires of such magnitude as may be propagated beyond the building of origin. The range in this additional cost has been found to be practically from zero up to 60% in certain classes of small modern cities.

It will readily be seen that this great cost of water-works chargeable to fire protection could be obviated to some degree if fire-proof construction were as good in the United States as it is in Europe, and to that extent the additional expense and use of necessary materials are waste.

The losses in natural resources due to fire waste are not the only preventable losses. Those resulting from earthquake are also great in amount and are equally preventable by the adoption of proper design and materials in the construction of public and private edifices. Recently, commenting on the most terrible earthquake disaster of modern times, that at Messina, Italy, the Italian Ambassador, Baron Edmondo Mayor des Planches, stated:

"My Government has many things for which to thank the people of America since the earthquake in Southern Italy besides the great monetary assistance and the sympathy we received. We are grateful for the scientific information we have received and the invaluable hints which have been given us by the United States Geological Survey. This information will be of more lasting benefit to Italy than even the great sums of money which have been sent to our people."

This statement of the Italian Ambassador was based on the information given to his country in a bulletin relating to the San Francisco disaster, the work of Richard L. Humphrey, Frank Soulé, and John S. Sewell, Members, Am. Soc. C. E., for the Structural Materials Division of the Geological Survey.

Mr. Wilson. Possible means of reducing the enormous waste due to fires, conflagrations, and earthquakes, are:

(1).—By tests and investigations to determine the relative fire-resisting properties of building materials, the relative rates of heat conductivity of such materials, and the development of systems of construction which offer the maximum resistance to fire. These tests should have in view the classification of the order of merit of the building materials, and the cheapening of cost of construction by the use of those best suited to the purpose, since cheaper materials are not often used through lack of knowledge of their availability and properties.

(2).—By the dissemination of information regarding the less inflammable materials of construction, their strength and durability, the methods of utilizing them in building construction, and the availability of the most suitable of these materials near the locality in which they are wanted.

(3).—By tests and investigations at the sites of such disasters as those at Messina and San Francisco, and by dissemination of information relative to the non-inflammable building materials, and the best method of designing and assembling them to fit them to withstand such shearing and torsional strains as they may be subjected to by earthquake.

(4).—By the enactment of building codes and the enforcement of the same with the view to securing more fire-resistant and more fireproof construction. In European cities the erection of wooden buildings is prohibited, and the oversight of brick, stone, steel, and cement construction is such as to diminish danger from fire due to defective flues, poor electrical wiring, etc., and to confine the fire to the building in which it originates.

It is evident that something must be done to stop the unnecessary waste of structural materials. Certain of these materials, such as wood and iron, are not inexhaustible by any means, but are even approaching exhaustion. In order to obtain the best use from these materials in the future, they must be used with a less lavish hand. Waste means increased cost in the very near future.

The known supplies of high-grade iron ores in the United States are estimated at 3 840 000 000 tons, and unless the present increasing rate of consumption is curtailed, they cannot last beyond the middle of the present century. There are, in addition, 59 000 000 000 tons, or nearly twenty times the amount of low-grade iron ore, which undoubtedly will be used when the conditions of the market warrant it. To increase the life of these iron-ore supplies, it is evident that the people of the United States must soon turn to concrete-making materials, brick, tile, and other clay products, and building stones, as substitutes for the more perishable timber and the more limited metal

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supplies. From the above a study of the causes of waste of structural materials is evidently of prime necessity. The first source of such waste has been shown to be fires. A second source, and one closely related to fire losses, is that due to waste of iron and steel placed underground in city water mains or in pumping plants, on account of fire and conflagration protection. A third source of such waste is that due to improper mining methods, whereby the whole product is not extracted; and an even greater source of waste is that due to improper or improvident methods of preparing the minerals for market both in shipment and in the metallurgical processes. Perhaps the largest waste of structural materials is that due to lack of knowledge of their strength. Engineers and architects adopt working stresses for concrete and metal construction from one-fourth to as high as one-eighth of the supposed working strength of the material. This means that from three to six times the necessary quantities of such materials may be used in structures, and, as a result, present systems of building construction are wasteful.

In order that this waste may be brought to the attention of the public and the law-makers, and in order that the discrepancy in cost between wood and less inflammable materials of construction may be reduced, thus encouraging the use of non-inflammable materials, the investigations on which the above data are based were undertaken. It is believed that through dissemination of information as to the local availability of cement-making materials, of gravel and sand suitable for concrete construction, of clay suitable for brick- and tile-making, and through tests and investigations which will show the most appropriate method of mixing and proportioning these materials, and of designing them with the minimum amount of each material which may suffice its purpose, will the cost of construction be reduced and the use of such materials be encouraged.

Within the past few years marvelous strides have been made in the substitution of iron and steel for wood, due to the investigations of engineers, physicists, and chemists into the properties of these materials and the great amount of attention given to their fabrication by manufacturers and architects. More recently the engineering and technical professions have advanced to a great extent the uses of cement in concrete manufactures, but in a vastly greater period practically nothing has been done toward ascertaining the physical and chemical properties and the better modes of manufacture and use of the products of clay and stone. With these objects in view, the Government, as the largest consumer of such materials, is undertaking such tests and investigations as may develop the most suitable of these less perishable building materials for each particular use and locality. These tests have in view the establishing of the physical properties of these materials, the suggestion of improved methods of manufacture with a view to

Mr. Wilson. economy, improved methods of mining and marketing in order to improve the quality, reducing the quantity and cost, and extending the life of such materials. The investigations include the assembling of information relative to the most fire-resisting and fire-proof forms of construction, the former for the prevention of conflagrations due to secondary or exposure fires and the latter for the prevention of the destruction of the buildings in which the fires originate. The tests also include investigations to prevent the loss of materials by the action of salt water on the Isthmus of Panama and in government and municipal sea-walls and piers, and also to prevent the loss of materials by the action of alkalies in the arid region, by electrolysis in cities, and by other destructive agencies. The average architect, engineer, or contractor does not take chances in the use of unfamiliar materials or designs, but confines himself to those of which he has knowledge as having proven moderately successful. Authoritative investigations, therefore, are in progress, with the idea of establishing the strength and properties of each of the materials under varying conditions in widely separated localities, and of pointing out the most economical material for each use.

Such of these investigations as bear on the construction of buildings to resist fire are divided into three groups. A series of such investigations is being carried on in the Structural Materials Division of the Geological Survey under the immediate supervision of Mr. Richard L. Humphrey, for cement, concrete, and reinforced concrete, and metal; under Mr. A. V. Bleininger for title, brick, and other clay products; and under Mr. E. F. Burchard for building stone. Under the Fuels Division, inquiries are being conducted, in co-operation with the National Board of Fire Underwriters, into the relative fire risk in the storage of such liquid fuels as gasoline, kerosene, and alcohol. In the Mine Accidents Division, an inquiry into the risks in storing and handling explosives is being conducted.

The Structural Materials Investigations already undertaken, and, in part, reported on, include fire tests of forty large panels representative of a section of a wall of a building. These panels were constructed of various building stones, brick, terra cotta, concrete building blocks, reinforced concrete, etc., and were subjected to the temperature to be expected in a conflagration. The concrete building blocks and the reinforced concrete panels were made of various grades of river and slag sands, various gravels, broken stones, cinder, etc. The brick specimens were of common, hydraulic pressed, and sand-lime brick. Careful record was made of the method of proportioning and mixing the mortar and the concrete materials. The panels were subjected to freezing and to hot water before and after the test and to drenching through the nozzle of a fire hose, when the maximum temperature had been reached. An effort was made to obtain the maximum tempera-

tures of conflagration conditions, namely, 1 700° Fahr. Thermo- Mr. Wilson.
couples were placed in the panels in order to determine the rate of temperature rise and the rate of heat transmission from the face to the rear of the panel. The temperature rise at the rear of the panel, in connection with the thickness of the latter, furnished information regarding the transmission of heat to the outer side of a building wall.

The work of the National Fire Protection Association should be known to every member of this Society. This Association includes as active members, the American Institute of Architects, the American Institute of Electrical Engineers, the American Water Works Association, and a number of other national technical engineering societies, fire underwriters' associations, city fire department associations, etc., and the writer believes that it would be to the advantage of this Society to be a member of that Association, in order that closer co-operation might be had along the lines of work on which both are engaged. The associate membership of that Association and its officers are chosen from that rapidly increasing group of professional men known as fire-protection engineers, many of whom are civil engineers, members of this Society, mechanical engineers, electrical engineers, architects, testing engineers, and the more advanced members and officers of the National Board of Fire Underwriters.

Through its committees and in co-operation with the Underwriters' Laboratories in Chicago, and the structural materials laboratories of the United States Geological Survey, valuable data are being collected and important reports issued on the best methods of fire-proof building construction. These reports are discussed at the annual meetings, and result in the preparation of specifications on such subjects as fire-proof construction, concrete and reinforced concrete in building construction, the proper materials to be used in the construction of gravity sprinkling tanks, the proper design of fire-proof window and door frames, elevator shafts, fire-walls, and other subjects of the kind on which the members of this Society who have to do with building construction should be informed.

The writer has trespassed on the time of the Society in bringing to its attention some of these matters, which may appear to be but distantly related to its functions, because it is his confident belief that only through an awakened and enlightened conscience, which can be most conservatively stimulated and wisely directed by engineers and architects, will the public, the producers, the manufacturers, and the governing bodies, embark on the reforms so sorely needed if building construction is to be placed on a more enduring and economic basis.

R. W. Lesley, Assoc. Am. Soc. C. E.—There is one point in the Mr. Lesley.
fire risk to which the speaker would like to call attention, that is, the legislation in reference to fires in the United States and in Europe.

In France, where he lived for some time, and was familiar with the

Mr. Lesley. law, the owner is responsible to the adjacent property holder for fire extending beyond his own building; in other words, if an owner chooses to build an inflammable frame structure, he takes the risk of burning down his neighbor's property, and he is liable for the damage, and this responsibility probably has a great deal to do with the forms of construction adopted. This policy has been adopted in many other countries of Europe, the speaker believes, and is a part of the law.

Mr. Humphrey. RICHARD L. HUMPHREY, M. AM. SOC. C. E.—All engineers, and particularly members of this Society, who have to deal with engineering construction, should give careful consideration to the facts presented in Mr. Wilson's discussion. The summary of these facts points out clearly the numerous annual losses in the United States, which are largely or entirely preventable, and the dangers of the destruction of property by fire which exist throughout the country.

The conditions which prevail in the United States are not unusual, and are the natural result in a country which has grown with great rapidity. The congestion of population has created problems which necessitate special legislation in order to correct the evils which have resulted from laxity and insufficient municipal supervision in building construction.

It is only after a city has outgrown the flimsy structures incidental and necessary to the settlement of new country that the congestion of population forces on the inhabitants the necessity for greater safety and provision against the dangers from fire.

The development of the building laws must necessarily be extremely slow, particularly because of the great difficulty in securing legislation which will correct the evils of the building laws caused primarily by the commercial influences which effect such legislation.

We are prone to look to Europe, to study the conditions there, and, in drawing comparisons, to calculate what our losses would be if we secured the same types of construction as obtain in European countries. This, of course, is very helpful, and is one of the means of drawing the attention of cities to proper legislation in a practical way. At the same time, it should be borne in mind that European cities are very old, and the congestion of population in the principal cities has been such as to necessitate, many years ago, such revisions of the building laws as would insure proper safety in those congested centers. In other words, European cities have been many years in obtaining perfection in their building laws. The United States is rapidly approaching, indeed it has approached, in many of its larger cities, such a condition in the congestion of its population as to make it absolutely necessary that steps be taken to prevent types of construction which are so dangerous as to form a constant menace of a serious conflagration. Many of these structures are fire-traps, constructed so flimsily that it is almost criminal to permit their existence.

Mr. Humphrey.

Anyone who was so fortunate as to investigate such great conflagrations as those at Baltimore and San Francisco is overwhelmed at the absolute worthlessness of the types of building construction which are in vogue in this country to-day. It is true that, after a fire has gutted a building, the walls and skeletons are found to be standing, and that the building has passed the conflagration satisfactorily. As a matter of fact, what remains of the structure forms a very small percentage of its value, the greater part of the latter having been lost in the destruction of the trimming and the various appurtenances which go to make up the complete building. In New York City buildings which are labeled fire-proof, and are apparently of reasonable fire-resistive construction, may be found to be surrounded by great areas of structures poorly constructed, with unprotected openings of plain glass which cracks at the first touch of the flame and enables a fire from a neighboring property to spread into the building and destroy its contents. Indeed, the intensity of some of the fires which occur in such buildings is so great as to destroy adjacent buildings which are reasonably fire-proof. Even the heat generated by fire in a building which is reasonably fire-proof is sufficient to destroy the integrity of the structure, because the building does not possess the requisite fire resistance. Structures should be designed so as to offer resistance required for the conditions. It is manifest that a warehouse in which large quantities of inflammable materials are stored must have greater protection against fire than a building which contains only a small quantity of such materials. Even where buildings are of the best fire-resistive type of construction known to-day, they are often destroyed by the conflagration of the fire-traps which surround them. For this reason, the building laws should be retroactive, in that they should compel the removal of all structures which are not of reasonable fire-resistive construction, especially in the congested centers of large cities. In the speaker's opinion, buildings should not be permitted to exist which are known to be fire-traps, and particularly should this be the case where the structure is used as a place of assembly for women and children.

In the matter of fire-resistive construction, it is a regrettable fact that most deplorable conditions exist in schools and places of public assembly, and, within recent years, there have been such terrible catastrophes as to horrify the entire nation, and yet these are only sufficient to stir up public sentiment for a brief period of time. As soon as the interest begins to wane, the public drops back into the same condition of indifference, and can only be aroused by a similar holocaust. If the building laws are to be adequately revised within a reasonable time, it is absolutely essential that the foremost societies of the United States, of which this Society should be a leader, should take an active part in the campaign, in order to provide sufficient pro-

Mr. Humphrey. tection against fire and thereby reduce the enormous annual losses. This Society has taken an active part in the work of conserving the nation's resources, and certainly there is not a more fruitful field for carrying on this work than in the attainment of structures of such a character as will offer adequate resistance to fire, thereby reducing to the smallest possible amount the enormous annual destruction of building material.

Mr. Whinery. S. WHINERY, M. AM. SOC. C. E.—This subject is one to which the speaker has not given much attention, but it seems to him that great importance may be attached to the particular fact referred to by Mr. Lesley, namely, the absence of personal responsibility on the part of the owner.

Doubtless, one reason why people are more careless in the United States than in any other country, is that, because of the very complete and efficient system of fire insurance, the losses through fire are not visited on the persons whose buildings are directly burned, but are practically shared by the whole community; and, while fire insurance is undoubtedly a very great benefit to the country at large, it may be a question whether it is an unalloyed blessing.

If the owner could be made to feel that if he puts up a cheap wooden building—a fire-trap, as it has been called—and that building should be burned down, a large part of the loss would fall on him, he would undoubtedly take more care to make it reasonably fire-proof; but, as long as he knows that he can insure the building for nearly its whole value, and thus compel the community at large to share the loss by fire, he will naturally be careless, and will favor cheap construction.

From this point of view, it might be wise, and it might also result in reducing the aggregate losses by fire, if insurance companies were prohibited from insuring any building for more than one-half of its value.

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PAPERS AND DISCUSSIONS

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IMPURITIES IN SAND FOR CONCRETE.

An Informal Discussion at the Annual Convention, July 8th, 1909.*

By Messrs. Sanford E. Thompson, R. W. Lesley, C. P. Howard, Richard L. Humphrey, G. S. Davison, S. Whinery, Charles M. Mills, and T. F. Richardson.

Sanford E. Thompson, M. Am. Soc. C. E.—During the last two or three years attention has been called to several cases where concrete failed to harden, and these failures could not be attributed to the quality of the cement. Some of these cases have been investigated thoroughly, one in particular to a point where quite definite conclusions were reached, and this will serve as an illustration of at least one of the causes of poor concrete. Mr. Thompson.

It has been proved conclusively that the sizes, and the gradations of sizes of particles of sand, affect the density, strength, and permeability of the mortar, and definite laws governing these relations have been framed, notably by Mr. Feret,† in France, which show that with sand having grains of known sizes the strength of the mortar may be estimated. These laws have been corroborated in a general way by tests in the United States and elsewhere.

In the United States, also, further tests have shown the effect of scientific methods of proportioning or grading the aggregates of the concrete upon its density and strength.‡

However, one frequently finds a sand which absolutely fails to obey the laws of gradation of sizes, or of density. Not only may the

*The discussion of this subject, for which no formal paper was presented, is printed in *Proceedings* in order that the views expressed may be brought before all members of the Society for further discussion.

† *Annales des Ponts et Chaussées*, IV, 1892.

‡ "Laws of Proportioning Concrete," by William B. Fuller and Sanford E. Thompson, Members, Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LIX, p. 67.

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mortar or concrete made from such sand fail to show the strength which would be expected, but it actually fails to harden, or the hardening which should take place within a week is delayed for perhaps two or three months.

Frequently, the sand which produces such bad results looks all right, and even those who have had much experience in concrete construction may be deceived by its appearance. Its mechanical analysis, that is, the gradation of sizes, may be good, and it may appear clean, and yet the quality may be such that it has to be absolutely prohibited from use in concrete, and, if used without previous testing, it will cause failure.

This does not disprove the laws referred to, but simply indicates that there is something further; that the laws of density apply to a clean sand, and that frequently it contains some other material which affects its combination with cement, either mechanically or chemically, or perhaps both. Such results, such failures of the mortar and concrete to set, show the absolute necessity, not merely of a careful examination of a sand which is to be used, not merely of a mechanical analysis to determine the sizes of the particles, but of a laboratory test, as thorough a test, in fact, as would be given to the cement itself.

In the special case referred to, a two-story machine-shop with concrete walls was being erected by local contractors, with no engineering advice except in the preparation of outline plans. The walls of the basement were 12 in. thick, those of the first story, 10 in. thick, and those of the second story, 8 in. The interior of the building was mill construction with timber beams and plank floors. One night, the walls being just above the level of the second floor, during a severe windstorm the building collapsed. An examination by the speaker, who was called on to make an investigation, immediately after the accident, showed that the concrete, although mixed in the proportions 1:2½:5, had failed to harden even in the basement.

The concrete of the entire building was so soft that, although parts of it had been laid for at least two months, a knife blade could be thrust into it, and it was even difficult to pick from the wall a piece hard enough to carry away as a sample. The appearance of the concrete was dark and dirty. There was a thin, hard skin on the outside, which had helped to deceive the contractors into believing that the material would eventually harden, although they were by no means satisfied with it.

Investigation showed that the concrete had been proportioned 1:2½:5 and mixed in a satisfactory manner. It was hand-mixed, but apparently well done, and tests of the actual proportions by analysis were considered unnecessary.

It was evident that the cause of the trouble lay in the materials. The last of the carload of cement had been used, only the empty bags

remaining; and because representative samples of this cement could not be obtained, the other materials had to be tested much more carefully than otherwise would have been necessary. Mr. Thompson.

The sand and gravel were taken from the site of the building when the cellar was excavated. The gravel was ordinary New England gravel, ranging from fine, that is ¼-in., up to particles perhaps 2 in. in diameter. It was fairly clean. Some of the pieces were slightly coated with dirt, but not more so than is almost always found in a gravel bank.

The gravel screened contained about 12% of sand finer than the ¼-in. size, that is, the screening was imperfect. However, this is almost always the case with screened gravel, so that in gravel concrete there is apt to be an excess of sand over the nominal proportions specified. In screening, a part of the sand, especially on wet days, is carried down with the larger stones.

The gravel was washed as gravel is usually washed in hand-mixed concrete. A hose was turned on the pile before the mixing was begun, and the fine material was washed down to the bottom of the pile and shoveled into a wheel-barrow with the rest. Then the gravel from the barrow was dumped on the mixing platform, and was again washed with the hose, the dirt simply flowing with the water from the surface to the bottom of the pile to be shoveled up and mixed with the gravel in the concrete. This is the ordinary but ineffective way of washing gravel in hand-mixing, unless special apparatus for washing is used.

The sand appeared to be good. In some places it was rather dark colored, but the most of it would pass an ordinary inspection, and would be called a sand of fairly good quality, certainly good enough for the work which was being done. The results of the mechanical analysis of the sand, given in Table 1, were above the average. Three per cent. by weight passed a No. 100 sieve, and about 25% was caught on a No. 8 sieve, but a closer examination of the bank showed considerable variation in the sand; in some places it was rather dark and reddish in color, while piles which had dried out looked "dead."

The representative of the cement company suggested a practical test which had never before come to the speaker's attention. Taking a double handful of moist sand from the pile, he allowed it to run between his hands as they were held with the thumbs up about 1 in. apart, at the same time moving his hands back and forth. Repeating this operation several times, always taking naturally moist sand from the interior of the bank, between the fingers of both hands there was collected a dark slimy substance which contained scarcely any grit. Some of this, scraped from the fingers and afterward tested by ignition, was found to consist almost entirely of vegetable matter.

A further examination of the bank showed that, in places where it had been cut to a vertical face for an excavation or for screening out

Mr. Thompson.

the sand, the rain had washed down from the surface soil a material similar to that which had collected between the fingers in the foregoing test, which formed a scum on the vertical face of the bank.

The surface soil was an ordinary, medium quality of loam, with, however, one or two dark, almost black, streaks in it, from ½ in. to ¾ in. thick.

All indications seemed to point to the cause of the trouble being vegetable matter in the sand, and vegetable matter which had apparently been washed down by the rains from the surface soil into the porous sand and gravel underneath, gradually coating the grains.

Tests of Sand.—Samples of the sand and stone were subjected to thorough tests. The mechanical analysis is shown in full in Table 1.

TABLE 1.—MECHANICAL ANALYSIS.

Analysis No.....	A-92.			A-96.		
Description.....	Average sand from excavation.			Reservoir sand.		
Date of collection..	May 5th, 1908.			1907.		
" " analysis...	May 8th, 1908.			May 9th, 1908.		
Percentage of moisture.....	2.6 before drying.			0		
Size of sieve.	Total weight passing.	Total percentage passing.	Percentage passing ¼-in. sieve.	Total weight passing.	Total percentage passing.	Percentage passing ¼-in. sieve.
Inches. / No.						
1.50 — 1½ in.	97.0	100.0	100.0	100.0	100.0	100.0
1.00 — 1 in.	97.0	100.0	100.0	100.0	100.0	100.0
0.50 — ½ in.	97.0	100.0	100.0	100.0	100.0	100.0
0.25 — ¼ in.	92.7	95.6	100.0	96.2	96.2	100.0
0.16 — 5 in.	81.8	84.4	88.4	88.9	88.9	92.5
0.0583 — 12 in.	56.5	58.2	61.0	68.2	68.2	71.0
0.0335 — 20 in.	39.5	40.7	42.6	48.2	48.2	50.1
0.0148 — 40 in.	14.6	15.0	15.8	17.0	17.0	17.7
0.0055 — 100 in.	2.6	2.7	2.8	2.3	2.3	2.4
0.0030 — 200 in.	1.8	1.9	2.0	1.1	1.1	1.1

Very fortunately, for purposes of comparison, the speaker happened to have in his laboratory a large sample of another sand which had been used satisfactorily in the construction of a reservoir* near Boston. This reservoir, because of its size and shape—100 ft. in diameter and nearly 50 ft. high—required special tests of the materials. The mechanical analysis of this good reservoir sand, shown in Table 1, was found to be almost identical with the sand used in the building which failed; moreover, the chemical composition was also practically identical, each containing about 75% of quartz.

Notwithstanding this apparent similarity, 1:3 mortar from the reservoir sand gave a strength of 272 lb. in 7 days and 332 lb. in 28

* *The Engineering Record*, January 12th, 1907, p. 32.

days, while the poorest sample of the sand in question gave an average of 20 lb. in 7 days and 75 lb. in 28 days. Mr. Thompson.

Volumetric Test.—To be sure that the bad quality of the sand was not due to the shape of the grains, which might prevent proper compacting, and, consequently, an excess of voids in the mortar, volumetric tests were made, and the density was found to be normal. The density of 1:3 mortar made with the sand in question was 0.679, and that of mortar made from the reservoir sand in the same proportion was 0.683.

Mica.—A little mica was observable in the samples. A careful examination, however, indicated that this was not sufficient to cause trouble.

Clay.—It has been sometimes claimed that, from a chemical standpoint, clay matter is injurious to sand. An examination of the chemical analysis showed, however, that the amount of clay, although large, was approximately the same as in the good reservoir sand, and, therefore, could not be considered as the cause of the poor quality.

Tensile Tests.—The tensile strength of the mortar from the sand in question, as already stated, averaged 20 lb. per sq. in. in 7 days and 75 lb. in 28 days.

The tests in different series using sand from different parts of the bank ranged, in the 7-day test, from 11 to 40 lb. To be sure that the cement used in testing or the laboratory methods were not at fault, tests were made in two different laboratories and with three different cements. Three different proportions were also used, namely, 1:2½, 1:3 and 1:3½. Specimens, for comparative tests in air, were stored in a moist closet, and in water. The results all checked so closely as to eliminate the question of cement or of manipulation.

Comparison of Moist and Dry Sand.—A comparison of 1:3 mortar made with the moist sand as it came from the natural bank and with the same sand after drying, showed some relative increase in strength due to the drying. In this series the average strength at 7 days of the sand direct from the bank was 8 lb. and of the dried sand mortar 23 lb.

Comparison of Natural Sand and the Same Sand after Washing.—A comparison of the sand in question before and after washing was as follows:

Unwashed.....	40 lb. at 7 days,	102 lb. at 28 days.
Washed.....	97 " " 7 "	196 " " 28 "
Standard	200 " " 7 "	275 " " 28 "

The grains of the washed sand were not thoroughly clean.

Microscopical Examination.—The microscopical examination of the grains of sand as obtained from the bank showed them to be covered with a dark brown coating which did not readily brush or wash off. The particles of the Waltham sand, on the other hand, were clean. A strong magnifying glass was found efficient in examining the grains for coating.

Mr. Thompson.

Washing Tests.—To examine and test the character of the silt in the sand, it was found necessary to remove the fine matter by washing.

To compare the results by washing and by screening out the silt through a No. 100 sieve, tests were made in both ways with the following results:

Worst sample of defective sand: 1.60% silt by washing; 1.14% by screening.

Average sample of defective sand: 2.46% silt from washing; 2.68% silt by screening.

Reservoir sand: 1.16% silt by washing; 3.66% by screening.

Although there is but slight difference between the results from washing and from screening, the chemical analyses indicate that the washing removes much more of the deleterious vegetable matter than the screening, so that this process should be followed when testing silt in sand.

It is noticeable in the above results that the poor sand gave a larger percentage of silt by washing than by screening, while the good sand gave a much smaller percentage by washing, indicating that in the latter case a large proportion of the very fine material was of heavier mineral origin.

The method used in washing to remove the silt was a very simple one, but was found to be more effective than several other more elaborate methods which were tried. The sand was placed in a large-mouthed quart bottle about half full of water and shaken thoroughly. The dirty water was poured off, and the operation repeated several times until the water was nearly clear. The wash-water was then evaporated, the residue thoroughly dried, and the loss on ignition, which determined the quantity of organic matter, was found.

Before igniting, the silt was passed through a No. 100 sieve to remove large particles of dirt which evidently would not be injurious if distributed through the mortar.

Chemical Analysis of Silt.—Both the poor sand in question and the good reservoir sand were washed and a chemical analysis was made of the silt, including a test of the residue by ignition. The analyses of the silt were as shown in Table 2.

TABLE 2.—Analyses of Silt.

	Percentage in reservoir sand.	Percentage in average sand, B, from site.	Percentage in worst sand, G, from site.
Moisture	7.80	8.24	13.07
Silica	54.12	45.32	34.94
Alumina	23.10	25.13	20.80
Oxide of iron	9.00	6.71	5.90
Lime	2.07	1.93	.....
Organic matter	2.50	11.86	26.33
Approximate percentage of clay	45.0	50.0	35.0

Mr. Thompson.

Inspection of these analyses shows but one notable difference: The organic matter in the silt from the reservoir sand was 2.50%, while in the silt from the poor sand it was very high. In one of the samples of poor sand the organic matter was 11.9% and in the other 26.3 per cent. When ignited, the organic matter in the poor sands gave off a peculiar odor of woody fiber, or leaf mould, thus indicating its probable vegetable origin. The good reservoir sand, on the other hand, gave no appreciable odor, and showed a reasonably small percentage of organic matter.

Reference has been made to the comparison of screening and washing. Tests by ignition show that, by washing, frequently double the percentage of organic matter is obtained than by simply screening.

It is noticeable that the silica is low, but it must be remembered that this is the analysis of the silt and not of the total sand.

Introducing Silt Into Standard Mortar.—To confirm still further the conclusion that the silt was the cause of the trouble, a mortar was made up of 1:3 standard sand with an addition of 1½% of silt based on the weight of the sand. The resulting tensile strength of the mortar was 29 lb. per sq. in. in 7 days and 107 lb. in 28 days, whereas the normal strength of the standard sand mortar was about 200 lb. in 7 days and 275 lb. in 28 days.

A similar test was made by introducing 1½% of silt into neat cement, and the resulting strength was about one-half that of the neat cement without silt.

Concrete Tests.—Specimens of concrete were also made, sand from different parts of the bank being used in different tests. The cement used in these tests was of a well-known brand, which was carefully tested to see that it was normal. The poor sand and gravel from the site of the building were used in the test as well as the good reservoir sand and gravel. The blocks were broken at the age of about 18 days at the Watertown Arsenal with the results shown in Table 3.

Hardening of Specimens.—Careful watch was kept of the specimens of concrete and of the mortar as they hardened, and pats of the concrete were also made. The mortar and concrete made with the good reservoir sand set up hard within 24 hours with a light gray color, while the other specimens remained so soft that they could not bear the pressure of the thumb nail for several days.

Conclusions.—As already stated, the tests indicated conclusively that the trouble with the sand was due to the vegetable matter which it contained, and that subsequent tests of other sand and examinations of structures indicate this to be a common cause for poor mortar or concrete. In many cases no failure results, but the concrete does not harden properly and never becomes as strong as it should.

The percentage of silt given in the chemical analysis of the silt appears to be large, being 11.9% in one sample and 26.3% in another.

Mr. Thompson.

When given in terms of the total sand, however, it is a very small percentage, because there is so small an amount of silt in the sand. Based on the weight of the total sand before washing, therefore, the percentage of silt as it comes from the bank is 0.27% in one case and 0.39% in another, an extremely small amount, and one which would not be indicated by any test of settlement in water or rubbing in the palm of the hand.

TABLE 3.—COMPRESSIVE STRENGTH OF CONCRETE, IN POUNDS PER SQUARE INCH.

Age, 18 days. Proportions, 1:2½:5.

Aggregate.	Helderberg Cement from Kendall & Wilkins.		Atlas Cement.	
	Individual.	Average.	Individual.	Average.
Sand A.....and Gravel E.	....	....	200	200*
" ¼ A + ¾ B " " E	818	.....	625	.....
" ¼ A + ¾ B " " E.....	692	755	749	687
" W..... " " E.....	....	....	653†	.....
" W..... " " W.....	....	....	1 505	1 505*

NOTES.—Sand A.—Worst sand from site of building.
Sand B.—Average sand from site of building.
Sand W.—Good reservoir sand.
Gravel E.—Screened gravel from site of building.
Gravel W.—Screened reservoir gravel.
* Only one specimen, but a good one.
† Only one specimen, and possibly slightly defective.

These and other tests indicate that there are two percentages of vegetable matter, which appreciably affect the quality of a sand: First, the percentage of vegetable matter in the silt, and second, the percentage of vegetable matter in the sand. Although the tests made thus far are too few to draw definite quantitative conclusions, it would appear that, in order to be injurious, the organic matter in the silt must be more than 10%, and, at the same time, the organic matter in the total sand must be more than one-tenth of 1 per cent.

Both these conditions are necessary, because it appears from tests that, in certain cases, the vegetable matter in the silt may be greater than 10%, but there may be so little silt in the sand that the organic matter in the total sand will be less than one-tenth of 1%, and the sand will pass the tensile test.

Whether the cause of the results is due entirely to chemical action or whether it may be due in part to mechanical action, the organic matter surrounding the grains so that the cement will not adhere, has not been determined. Most probably the cause is chiefly chemical, but in a small measure also mechanical. The subject appears to be of sufficient importance to warrant further and thorough investigations.

Mr. Thompson.

In conclusion, special stress should be placed on the necessity in concrete work of thorough tests of the sand. It is not merely necessary to examine sand with the eye; it is not sufficient to test it by rubbing it in the hands; it is not enough to make a mechanical analysis and to determine the sizes and the gradations of the particles, but, in every case, unless the sand is from a bank of known good quality and has been previously tested, as careful tests as are required of the cement should be made. Probably the best one is the ordinary tensile test advocated by the Society's Special Committee on Concrete and Reinforced Concrete, which requires that:

"Mortars composed of one part Portland cement and three parts fine aggregate by weight when made into briquettes should show a tensile strength of at least 70% of the strength of 1:3 mortar of the same consistency made with the same cement and standard Ottawa sand."

In case a sand must be used immediately, with no time to make tensile tests, or in case special investigations are needed to determine the causes of poor quality, the washing test and determination of the organic matter are of special value. The mechanical analysis, which shows the proportions of the grains of different size, is also of great value as indicating the comparative value of different sands which are free from organic matter.

There is one further thought which might be brought out. These impurities have been spoken of as dirt and matter which might be washed out. One sample of sand which was sent to the speaker from Philadelphia, had a great deal of dirt in it, that is, it had much more fine material than the defective sands which were investigated, and yet washing it increased the strength of the mortar instead of decreasing it. The fine material was found to be not organic matter, but of mineral composition.

Mr. Lesley.

R. W. Lesley, Assoc. Am. Soc. C. E.—Mr. Thompson's contribution is very interesting. In the light of the recent report of the Special Committee on Concrete and Reinforced Concrete, of this Society, it rather emphasizes the principle of "locking the stable door after the horse has been stolen;" in other words, the building had fallen before the exhaustive examinations of sand were required. On the other hand, the speaker believes that the report contains, in one paragraph, what is more germane to the subject of concrete and reinforced concrete than any other lesson, namely, that the sand shall be tested in the same way that the cement is tested. The mortar must be a standard mortar. This report provides a means to determine that question.

At the present moment there is no doubt that in every large cement manufacturing plant in the United States there are many inspectors who are testing cement prior to its shipment to various parts of the

Mr. Lesley. country. The speaker happens to know that, to-day, in one mill, there are no less than twenty or twenty-five men engaged in testing and inspecting cement. One part of that cement is to be used to three parts of sand, and yet the speaker does not know of anyone anywhere who is testing the sand to be used in the same manner that the cement is being tested.

The engineer requires that cement shall be tested in a dozen different ways, to show that it meets the requirements of the specification, and no cement manufacturer, under existing conditions of inspection at the mill, can produce a cement that will not be actually and scientifically right; and yet that cement may be used, in the proportion of one to three, with a sand that no one has tested, no one has examined, and about which nothing is known.

Mr. Thompson's paper brings us face to face with a state of affairs which has been going on for many years, and, by making a requirement that all sand shall be tested in the same manner that cement is tested, the Committee on Concrete and Reinforced Concrete has certainly taken a great step forward.

In the recent report of the Building Code Commission of the Board of Aldermen of New York City, a step farther has been taken. Whereas, to-day, cement only is tested, under the report of the Committee on Concrete and Reinforced Concrete, both the cement and the sand will be tested. But there is still another step: Suppose one has good cement and good sand, and yet the building fails. That failure may be due to the fact that the contractor has used seven or eight, or even ten parts of sand, and again there is an opportunity "to lock the stable door after the horse has been stolen." In this particular report of the Building Code Commission, it is provided that the sands represented in the mortar shall be inspected from time to time, and that the mortar from the mortar box shall have 70% of the strength of the same cement, used with standard Ottawa sand. In other words, it places the responsibility for the mortar which binds the concrete, or which binds the masonry, on the men who are actually providing that mortar.

The cement is to be tested, the sand is to be tested, and the mortar is to be tested, and the speaker believes that this is a most important step forward. He also believes that, this country being one of the most progressive in the use of concrete, the day is not far distant when the American, instead of whittling timber with a knife, will whittle concrete with a shovel; that the same cleverness which, heretofore, has enabled American genius to make its way forward, will enable it in the future to whittle with a shovel, and to do things with this plastic material which to-day seem impossible.

Recently, at a meeting of one of the scientific societies, one man read a paper on the use of concrete with nails as reinforcement.

Another engineer stated that he had made some concrete and had reinforced it with jackstones; and that statement was again made ridiculous by a man who had used old hoop-skirts as a reinforcement, and if American genius has gone so far in the use of concrete, there seems to be no limit to what it will do in the future. Mr. Lesley.

C. P. HOWARD, M. AM. SOC. C. E.—The speaker has recently seen some specifications in which there was a provision that the stone used should be free from oil, and understood that the parties using these specifications went so far as to refuse stone from certain quarries on account of the oil which it contained. Such a precaution would probably not occur to most engineers. It emphasizes the importance of the most careful examination of all three ingredients, namely, cement, sand, and stone, and also of actual tests of concrete briquettes made from the same materials used in the structure. Mr. Howard.

RICHARD L. HUMPHREY, M. AM. SOC. C. E.—There has been a decided change in the requirements for sand, and this is necessary, because, to-day, the character of the sand used in cements, mortars, and concretes is of the highest importance since it has a vital effect on their strength. Specifications which require the sand to be clean and sharp are wholly inadequate, and yet there are many engineers who pass on the quality of sand by its appearance, thus complying with this specification. The fallacy of thus judging it is readily illustrated in an examination of sands from different parts of the country, such an examination showing how easily the eye can be deceived as to its quality. Sands have been sent to the speaker from various parts of the country, and he has had the opportunity, in a number of cases, of noting how easily it is possible to be deceived. As an instance, two sands from the Isthmian Canal may be given. The one which complied with the specification by being clean and sharp, and apparently the better of the two, proved by test and careful examination to be inferior to the sand which was not sharp and appeared to be dirty. The physical tests were the means of determining the relative quality of these two sands. As a matter of fact, the poorer sand was the cheaper as it was more readily accessible for the work in question. This, the speaker believes, is one of the many illustrations of the value of making physical tests of sand before passing on its quality. Elaborate tests of cement are made, and just as thorough tests of the sand should be made, because its effect on the strength of concrete is as great, if not greater, than the cement which binds it together. Many sands which appear to be sharp and of a fine quality are of uniform size and contain so much fine material that they produce very slow-hardening mortars, thus interfering with the stability of the structure in which they are used. Mr. Humphrey.

There is no doubt that a great deal of the inferior sand used to-day is the cause of failures. It may be stated that the use of this sand

Mr. Humphrey.

is in a large measure due to a reasonable ignorance on the part of engineers and others in charge of the work. It is highly important that the engineer should be alive to the necessity of testing the sand just as rigidly as he tests the cement, thereby securing a better grade and eliminating many troubles which may arise from the use of improper material. The report of the Society's Special Committee on Concrete and Reinforced Concrete, in requiring a mortar-box test, marks an important step forward.

The fine material found in sand, which is generally referred to as loam, should be properly called silt, because it generally consists of very finely-divided material of the same character as the sand itself. In the investigation of sands from all parts of the country, by the Structural Materials Testing Laboratories of the United States Geological Survey, it has been found that the presence of this fine material, if it is of a granular character, proved beneficial, as it increased the density of the mortar or concrete, and consequently its strength. In an effort to increase the impermeability of mortars and concretes, so-called colloidal clays have been used. A fine sand is always used with the idea of filling voids, and the speaker believes that its character is immaterial, if it is granular and is sufficiently fine to fill the voids properly. Indeed, where the so-called colloidal clays seem to produce a greater density, it has been found that a very finely-divided sand would produce the same effect. On the other hand, there are sands and gravels which contain an excess of finely-divided material which it is necessary to screen out in order to get the requisite density and strength. In the study of sands referred to, an appreciable quantity of organic material is rarely found. Where sand or gravel is not well graded, or concrete is not properly proportioned to secure maximum density, a finely-divided material or a little extra cement itself may be added with beneficial effect. Where the sand or gravel naturally possesses the requisite amount of fine material to fill the voids, the addition of more material of this character would tend to reduce the strength of the mortar and prove otherwise detrimental. Again, a large percentage of finely-divided material will produce mortars and concrete which set very slowly, and this is a decided disadvantage. In this discussion material finer than will pass a No. 200 sieve is referred to as silt. Many natural sands contain 6 or 8% of a very fine material, which, by reason of filling the voids, adds materially to the strength of the resulting mortar and concrete, and gives rise to the impression that small percentages of finely-divided material are beneficial to the mortar or concrete. If the mortar or concrete is granular, and the fine material does not occur as a coating on the particles of sand, and is not in too great excess, there is no doubt that it will be beneficial. On the other hand, fine material which is of a flaky character and coats the particles of sand or gravel, has a decided weakening action, and is objectionable.

The use of clays of various kinds is also objectionable, because in order to mix them with the cement properly, they should be thoroughly ground and dried to an impalpable powder. Otherwise, they will have a tendency to ball up, and the result will be little nodules of clay in the mortar or concrete. Mr. Humphrey.

G. S. DAVISON, M. AM. SOC. C. E.—The importance of using proper sand for cement mortar has long been recognized. Some fifteen or sixteen years ago the speaker drew up specifications for cement mortar to be used in the foundations of a large industrial establishment, and great care was taken to specify that the sand should be sharp river sand, such as is found in abundance in the neighborhood of Pittsburg. Mr. Davison.

This sand was shipped to the point of construction by car, and at considerable expense to the contractor for freight. After a few days it was discovered that the mortar was not setting properly, and, on making a series of investigations, it was decided that the trouble was caused by the sand. It was just as specified, but the mortar made from it would not stand the tests. As a temporary expedient, it was agreed that, for a few days, and in some unimportant foundations of the building, the contractor would be permitted to use, in the mortar, bank sand which was being excavated for the foundations for the building. Much to everybody's surprise, the mortar made with this bank sand was satisfactory in every particular.

S. WHINERY, M. AM. SOC. C. E.—The word "sharpness" is frequently used in speaking of the qualities of sand, and perhaps it is worth while to call attention once more to the fact that it is an indefinite and even deceptive term. This, the speaker believes, has been pointed out a number of times in the *Transactions* of the Society. The test usually applied to determine the sharpness, that of grinding the sand between the fingers, is, to say the least, not decisive. The rounded and polished grains of a beach sand, if of a certain size, seem by this test to be decidedly sharp. If these same grains be crushed to a comparatively fine powder and rubbed between the fingers, they will give the impression that they lack sharpness, although the individual grains are angular, with very sharp edges. Mr. Whinery.

It is undoubtedly true that the character of the different kinds of sand has not been investigated sufficiently in connection with the making of mortar and concrete. There may be great differences in the character of sands, which, in most respects, appear to be alike. Some have a greater degree of what has been called the quality of adsorption, and the cement seems to take a stronger hold of them than it does of other sands which are apparently just as good. This quality of the surface of sand grains deserves more careful investigation than it has yet received. Until engineers are able to drop the indefinite term "sharpness," in specifications for sand, and to substitute for it some accurate scientific description of the sand wanted, unsatisfactory results may be expected sometimes.

Mr. Mills.

CHARLES M. MILLS, M. AM. SOC. C. E.—It may be of interest to state that the sand to which Mr. Thompson refers was taken from the excavation for the subway on Market Street, Philadelphia, Pa., from the intermediate and lower strata, the cleanest being selected for the work.

The large proportion of silt was regarded with suspicion, and its use in concrete, except in unimportant details, resulted only after thorough tests, extending over 90 days. The material was hauled to a yard, passed through ¾-in. screens, and used without washing; it contained a good proportion of coarse particles, well graded from coarse to fine. Daily tests were made, in order to control the quality, the proportion of silt varying from 6 to 29 or 30%, which was determined by the matter not settling in 45 sec. when a sample was agitated in water, the sample being successively agitated in fresh water until the water was clear. The proportion of silt averaged about 21½%, the upper limit of 30% being unusual, and representing the approximate limit for rejection. The silt was practically impalpable, and analysis showed that it consisted usually of fine sand and disintegrated mineral matter in about equal proportions. The sample furnished to Mr. Thompson contained 0.4% of organic matter which did not seem to affect the strength in mortar briquettes.

Briquettes in the proportion of 1:3 gave better tensile strengths than those obtained from any other of the coarse sands or fine gravels offered for use, except in the case of a certain river-washed silicious sand which was remarkably clean and well graded from coarse to fine. With this exception, the tensile strength of 1:3 briquettes made with the excavated gravel was in excess of that obtained from briquettes made with all other materials, except mixtures of stone dust and stone grit. A few typical tensile tests, given in Table 4, show the results with standard quartz sand, the river-washed sand referred to, and the excavated gravel, in which the content of silt is stated.

The tests in Table 4 are in general agreement with many others made during the progress of the work, both in the values obtained and in the vagaries with respect to the effect of the varying proportions of silt, and, in considering the latter, the constituents of the silt should be borne in mind.

The silt comprised the fine material so desirable in producing dense and water-tight concrete, and, at the same time, the material in which it occurred made tenacious mortars.

The tests on 6-in. cubes revealed the same vagaries with respect to the effect of more or less silt in the excavated gravel, the tests and experience with the concrete in which this gravel was used confirming the favorable conclusions reached from the tensile tests respecting the value of the gravel for use as the fine aggregate. The presence of so much fine material made it necessary to exercise special care in

Mr. Mills.

TABLE 4.—TESTS ON STANDARD BRIQUETTES, MADE WITH PORTLAND CEMENT, IN PROPORTIONS 1:3.

Material.	7 days.	28 days.	3 months.	6 months.	Maximum silt. %	Minimum silt. %	Average silt. %	Remarks.
Standard quartz sand.....	(33) 213	(33) 278	(27) 327	(18) 390	.....	.....	.....	Values given are averages from the number of briquettes in parentheses.
Excavated gravel.....	(33) 291	(33) 364	(27) 427	(18) 475	29.00	6.80	18.30	
River-washed gravel.....	(12) 276	(12) 405	(12) 472	(6) 443	4.00	Trace.	1.62	
Excavated gravel.....	(3) 297	(3) 396	(3) 401	(3) 451	29.00	.....	.....	Values given are averages from three briquettes.
Excavated gravel.....	(3) 311	(3) 393	(3) 453	(3) 469	.....	6.80	.....	
River-washed gravel.....	(3) 311	(3) 441	(3) 439	(3) 439	4.00	.....	.....	
River-washed gravel.....	(3) 254	(3) 413	(3) 517	(3) 447	.....	Trace.	.....	

Mr. Mills.

cleansing the scum from the surfaces of previously deposited concrete in making joints. Briquettes made with cement and the pulverized dried scum gave very small tensile strengths.

It is desired, incidentally, to emphasize the necessity for the examination of the properties of silt in any given case, since it may either be a source of dangerous weakness in mortar, or a valuable constituent in gravel for concrete, when water-tightness is desired.

One purpose in introducing the foregoing is to invite a discussion of the following question, which is asked with full appreciation of the laboratory researches and the experiments with colloidal clay and other materials in reducing the permeability of concrete: When water-tightness, or the reduction of permeability in concrete is necessary, and the available materials for concrete do not contain in themselves a sufficient quantity of the fine material required for this result, is it wise to introduce a fine material in the intermediate aggregate, or is it better to increase the quantity of cement?

In considering the foregoing, it is desired that the practical features of the case be considered from the standpoint of the engineer who aims to control the quality of the materials, and to see that such materials are properly used. It is also understood that the most judicious proportioning of the coarse and fine aggregates will be made in any given case and that the proportion of cement will produce the desired structural strength. It is obvious that local considerations of available supplies will affect the decision, but it is desired to assume that the requisite materials are available at reasonable, or what may be termed balanced, prices. It is also to be assumed that the materials are mixed by machine, according to approved methods, and that the question is intended to apply to natural or ordinary commercial material, and not to patented or secret compounds.

Mr. Richardson.

T. F. RICHARDSON, M. AM. SOC. C. E.—The statement has been made that fine sand or silt tends to increase the density of mortar. This may be true for lean mortars where 1 part of cement is used to 4 or 5 parts of sand, but the speaker does not think that it is true for richer mortars such as are commonly used in American practice. It certainly is not true for mortar of 1 part of Portland cement to 2 parts of sand.

During the construction of the Wachusett Dam of the Massachusetts Metropolitan Water-Works, extensive experiments were made to determine the effect of using sands of different sized grains, and various combinations of sand, on percolation and on the tensile strength of mortars. Three grades of sand, called fine, medium, and coarse, were used in the experiments; the fine sand all passed a No. 100 sieve, the medium sand all passed a No. 30 sieve and was held on a No. 100 sieve, and the coarse sand all passed a No. 8 sieve and was held on a No. 30 sieve, these sieves having 10 000, 900, and 64

meshes, respectively, to a square inch. All three sands had practically the same percentages of voids, ranging from 41 to 42%, and the fine sand had practically the same percentage of voids and the same mechanical analysis as the cements used in the experiments, which were standard American cements. Mr. Richardson.

Experiments to determine the percolation through, and the tensile strength of, the mortars, were made with the fine, medium, and coarse sands, and with nineteen mixtures of these sands, the mortars experimented on consisting of 1 part of cement to 1, 2, 3, and 4 parts of sand by measure. A mixture of 75% of coarse sand and 25% of fine sand gave a sand having 31% of voids.

As illustrating the density of the mortars, it was found that a 2:1 mortar, made from two barrels of coarse sand and one barrel of cement, gave 2.4 barrels of mortar, while, if two barrels of fine sand were used, 3.0 barrels of mortar were obtained. All mixtures of the fine, medium, and coarse sands gave more mortar than the coarse sand, the 75% of coarse and 25% of fine sands giving 2.53 barrels of mortar for the 2:1 mixture. This was to be expected, as the 41% of voids in the coarse sand, being large, were filled by the cement, while the 42% of voids in the fine sand were not filled by the cement, as the fine sand and the cement had grains practically of the same size.

In testing for percolation and for the tensile strength of the mortars, it was found that mortars of coarse sand with no admixture of fine or medium sands gave practically the least percolation and the greatest tensile strength, with 1 part cement to 1 part or 2 parts sand. For mortars of 1 part of cement to 3 parts of sand, the mortar made of 50% coarse, 25% medium, and 25% fine sands gave slightly better results than that made with 100% of coarse sand, both as to percolation and tensile strength, and this was also true to an increasing extent for mortar of 1 part of cement to 4 parts of sand. The mortars of fine sand, whatever the proportions of cement might be, in all cases gave the greatest percolation and the least tensile strength.

The sands used in the above experiments were almost entirely silica and contained little or no alumina. The pressure was 75 lb. per sq. in.—equivalent to a head of about 170 ft.

In coarse sand the particles themselves are impervious to water, and the fine cement fills the interstices between the grains for mortars consisting of 1 part of cement to 2½ parts of sand or richer. For leaner mortars, consisting of 1 part of cement to 4 or 5 parts of sand, such as were experimented with in the French tests by M. Feret, which have been referred to, a mixture of fine or medium sand is desirable to help out the cement. Such mortars, however, are rarely used in American practice. M. Feret's tests indicated that the best sand was a mixture of coarse and fine sands, without any medium sand. The experiments referred to by the speaker indicated that medium sand mixed with coarse sand is more desirable than fine sand.

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AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

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HAS EXPERIENCE DEMONSTRATED THAT THE OILING OF ROADS IS THE MOST SATISFACTORY OR ECONOMICAL METHOD OF PREVENTING DUST AND PRESERVING THE ROAD SURFACE?

An Informal Discussion at the Annual Convention, July 8th, 1909.*

By Messrs. S. Whinery, and G. S. Davison.

Mr. Whinery.

S. Whinery, M. Am. Soc. C. E.—We have now had sufficient experience with the use of oils and tars for dust prevention and road preservation to make it worth while to take stock of the facts available, to consider their import, and to form some tentative conclusions as to the efficiency and economy of the process. Only one other process or method of accomplishing the same results is available for comparison, and that is watering or sprinkling. It is true that modifications of these two leading methods have been used more or less. Among these is the use of a solution of some deliquescent salt in water, and an emulsion of oil in water; but while these have each been reported as having given satisfactory results in a number of cases, it can hardly be claimed that their use is general or extensive, and the verdict of experience is mostly against them. In any event, it may be said that they are only modifications of the two primary methods, and there is good reason to believe that none of them will be found more satisfactory or, on the whole, more economical than the use of oil or water alone.

The discussion, therefore, is narrowed down to a question of oil or water. It may also properly be confined to the treatment of ordinary broken stone or macadam roads, in the eastern section of the

*The discussion of this subject, for which no formal paper was presented, is printed in *Proceedings* in order that the views expressed may be brought before all members of the Society for further discussion.

Mr. Whinery.

United States, which carry a sufficient quantity of travel to make them very dusty in dry weather, and to cause them to wear and disintegrate rapidly, unless they are properly cared for.

It is known from long experience that the dust may be kept down by sprinkling with water, and that if the body of the macadam be kept properly moist, abrasion from travel will be largely, and that kind of disintegration called "raveling" almost wholly, prevented. This is particularly true when the bulk of the tonnage passing over the roads consists of motor-driven vehicles, as the action of their rubber-tired wheels is quite different from that of steel-tired wheels. In recent years the opinion has become quite general that equally good, or better, results can be accomplished more economically by applying oil or tar to the surfaces of roads. The question under discussion, therefore, may be resolved into this: Can dust be avoided, and can the wear and raveling of macadam roads be prevented more satisfactorily and more economically by the use of oil than by the use of water? To consider this question fairly, it must be assumed that each method shall be intelligently and skilfully used, or, in other words, that each shall be given a fair chance in the contest between them. It would be obviously unfair and unprofitable to judge the merits of either from the results of average practice, where that average is largely affected by ignorance or carelessness. In most cities and towns street or road sprinkling is little short of a delusion and a snare for the municipal funds. Little attempt has been made by municipal engineers to study, systemize, or regulate it in a scientific manner. Statistics of its unit cost, and of the quantity of water required, have been difficult to get, because sufficient attention has never been given to the work to insure the collection of accurate data. In the case of road oiling, the absence of knowledge and skill, and not infrequently of ordinary good sense, has led to a multitude of failures and to the waste of large sums of money. But in this work, thanks largely to the United States Office of Public Roads, and the different State road departments, there are quite full and reliable cost data, though not very many of these data relate to what is now considered the best practice.

In favor of street or road watering, it may be said that, if properly and intelligently applied, it will effectually suppress dust, and will prevent the disintegration of the road material by raveling. It is, in fact, almost unanimously agreed among good road engineers that raveling is caused by the absence of a proper amount of moisture in the macadam material. It is doubtless true that water does not prevent surface abrasion as effectually as oil, but with the large ratio of rubber tires now found on the roads, abrasion in the true sense of the word is not a very important element of road deterioration.

The most serious objection to watering is that when improperly

Mr. Whinery. done, that is when the street is flooded with a surplus of water, it is likely to be sloppy and disagreeable for pedestrians, and the wheels and bodies of vehicles may be spattered with mud, which, however, is readily removed by washing with water alone. One advantage of some importance is that the material for watering streets is always at hand, and the plant required is simple and usually available at any time, so that the roads can be sprinkled at any time during the year when it may become necessary. It is also an advantage that travel need not be interrupted or interfered with, while the water is being applied, or afterward, and that no special preparation of the street surface is necessary.

Oiling is as effective as watering for suppressing dust, if the whole road surface is oiled. But usually, because of the cost, only a strip in the middle of the road is treated, leaving untreated strips along each side, which may be troublesome sources of dust in dry weather if the travel is heavy. Oiling cannot be done properly unless the road be perfectly dry and the weather favorable, and for this reason it is thought expedient, if not absolutely necessary, to postpone the treatment until quite late in the season. In the meantime people must endure the dust or resort to watering for from one to two months in the spring. The surface must be carefully cleaned and prepared for the reception of the oil, and, after it is applied, the road, or the oiled part of it, must be closed to travel for from two days to a week. Even after it is opened to travel, the surplus oil still on the surface makes it very disagreeable for pedestrians for several more days, and the free oil or oil-saturated dirt is thrown by the wheels over the vehicles and the clothes of the occupants, and these greasy stains are very troublesome to remove. In the autumn and winter season following, these oiled surfaces produce an abundance of oily slime and mud, creating a condition for which the word "nasty" is an inadequate term, and since the remaining oil is sufficient to make this mud more or less cohesive and water-resisting, it is not carried away by the rains, and continues through most of the winter.

The extent to which oil preserves the road surface from wear and disintegration is not yet definitely determined. As long as the oiled surface remains unbroken there is practically little loss by abrasion. That oiling does not prevent, but rather promotes, a kind of disintegration analogous to raveling, on heavily traveled roads, is evident to those who have observed the effects carefully. This result might be expected and predicted on theoretical grounds. Raveling is caused by the drying out of the body of the macadam. The oiled surface, compressed by travel, forms an impervious coating over the road through which the rain cannot penetrate. The result is that the macadam gradually loses its moisture and consequently its cohesion, and the dense surface coating is broken up and comes off in patches,

carrying with it a stratum of the distintegrated macadam surface. These patches enlarge rapidly and others appear, so that roads carrying a heavy travel during a dry season are likely to become in bad condition before the winter begins. Mr. Whinery.

This statement may seem overdrawn to many, but it correctly represents conditions actually observed during the season of 1908 on first-class macadam roads well treated with the best quality of oil, in an eastern city. The speaker has no hesitation in saying that at the end of the season the physical condition of the oil-treated sections of these roads was not better than the condition of adjacent sections of the same roads, which were sprinkled more or less ineffectually.

Even if it were true that oiling is, on the whole, rather more efficient than watering, the comparative cost of the two processes must be given proper consideration before a final decision is arrived at.

Reliable and accurate data relating to the cost of street sprinkling are rather meager. Even where prices and areas are given with some care, so many other factors and conditions are usually omitted that it is impossible to deduce satisfactory unit costs. A careful study of the data available, supplemented by careful estimates of what the cost should be, leads to the conclusion that on macadam streets carrying a fairly heavy travel, the work as now usually done does not cost more than 2 cents per sq. yd. per season. In a number of cities where the work has been done with more than the usual care, the cost has been below that figure, which includes the cost of the water used, charged at $75 per million gallons. Street sprinkling, however, is very seldom properly done. The sprinkling is too infrequent, and the quantity of water applied at each sprinkling is too great. Where the surfaces are ordinarily sprinkled twice daily, they ought to be gone over three times daily in order to secure the best results; but, on the other hand, with proper care to avoid waste, the quantity of water used might be materially reduced. One may often see twice the quantity of water actually necessary being discharged over the street, and the surplus half running into and flowing away along the gutters. It may be safely stated that, with the more frequent sprinklings needed and with the work done with the same care and skill necessary for successful road oiling, a road of the character just considered can be watered so as wholly to avoid dust, and to preserve the road itself at a cost not exceeding 3, or, at most, 3½ cents per sq. yd. per season. It should not exceed the lower figure under efficient management.

In a large number of cases the cost of oiling roads has been accurately recorded, and the unit costs have varied as widely as the character and quality of the work. In what is now regarded as the best practice, using a high-grade asphaltic residual pitch, with the necessary preliminary preparation of the road surface and the proper

Mr. Whinery. application of the oil, a cost of from 5½ to 6 cents per sq. yd. may be regarded as reasonable for a single effective application. Such an application made between the middle of June and the first of July should be effective, on the class of roads being considered, until the end of the usual sprinkling season, but its effect will almost wholly have disappeared before the next spring, when the treatment must be repeated *de novo*.

During the early part of the season, say from the first of May, when sprinkling is usually started, to the middle of June, which is about as early as oiling is practicable, sprinkling must be resorted to or the dust endured. Disregarding this, however, and calling the cost of efficient sprinkling 3½ cents per sq. yd. per season, and the cost of oiling 5½ cents per sq. yd. per season, it appears that the oiling costs 57% more than the watering. The speaker does not believe that experience has yet demonstrated, or will in the future demonstrate, that, everything considered, oiling is 50% more efficient or satisfactory than sprinkling, when each is equally well done.

If the statements and figures here presented are reasonable and sound, the question seems to have been answered, and this other question is suggested: Is road oiling anything more than a popular fad of the day, unworthy of the further serious attention of engineers?

Engineers are all looking for the advent of some new kind of road, which, while not too expensive for use at least on the more important roads, shall be more durable than macadam and shall not generate dust. The solution of the problem does not seem far off. Such roads may be truly economical even though their first cost seems high; they will certainly be cheaper in the end than oiled macadam roads. The payment of 6 cents per sq. yd. per annum for oiling represents an investment, at 6%, of $1.00, and it is reasonably certain that less than $1.00, added to the first cost of the plain macadam, would provide a road which would not need oiling. The wise course, then, is to give more attention to constructing new roads which will be practically dustless, and to be content with the cheaper and quite as effective method of watering to combat the dust on the old roads.

Mr. Davison. G. S. DAVISON, M. AM. SOC. C. E.—The speaker would like to ask Mr. Whinery what part of the cost of oiling the roads in question, was freight on the oil. In certain cases this freight item might make the use of oil prohibitive.

The greater part of the oil used for this purpose is from Southern refineries located near the seacoast, and is sent to Boston, New York, and Philadelphia, and such points, with very low freight charges. Such oil will come within reasonable figures, when the cost of reshipment is low. Therefore, unless the oil for roads in the interior of the United States can be obtained from refineries close at hand, the oiling of roads must be confined to points near the seacoast.

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PAPERS AND DISCUSSIONS

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CONCRETE PILES.

Discussion.*

By Messrs. Morton L. Tower, L. J. Mensch, Charles H. Higgins, Maxwell M. Upson, and Thomas C. Desmond.

Mr. Tower. Morton L. Tower, M. Am. Soc. C. E. (by letter).—Owing to the continual rise in the price of timber, as well as on account of the indestructibility of concrete piles, the latter are sure to have an increasing range of application. Even in the locality of San Francisco, with the fine Pacific Coast pile timber within a radius covered by low ocean freight rates, the concrete pile has found a demand, and its design is a matter of interest and a subject of study for engineers.

For lengths of 60 ft. and less, the design of reinforced concrete piles contains no unusual features, and numerous examples have been illustrated and described in the technical magazines and society proceedings. For piles more than 60 ft. long, the weight and strains due to handling render excessive reinforcement necessary, and, thus far, such piles have had few, if any, practical applications. In all probability, a long pile could be designed in the form of a hollow concrete cylinder with longitudinal diaphragms, which would be considerably lighter than a solid pile.

Metal points on "made-before-driving" piles seem to be superfluous. Shoes were once deemed essential on wooden piles, but are seldom used at the present time. For the "made-in-place" type, using some form of light metal casing driven into the ground, the metal point is of course essential.

Piles with square ends are much more easily kept in line while driving, while the points add little if any to the rate of penetration. The advantages of a square end for bearing power are very evident.

*This discussion (of the paper by Howard J. Cole, M. Am. Soc. C. E., printed in *Proceedings* for May, 1909, and presented at the meeting of September 1st, 1909), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

Mr. Tower. For these reasons, concrete piles for general use should be designed with square ends; for stiff clay soils, resulting in hard driving throughout the entire penetration, the pointed end may be of sufficient advantage to warrant its use.

The advantages of the water-jet for placing concrete piles are very evident. Probably this method of driving has been developed more fully at the jetty at the mouth of the Columbia River than anywhere else.

The details of the apparatus and methods used there, with very few changes, would apply to the handling and placing of concrete piles. Briefly, this consists of two jets, one on each side of the pile, the water emerging from 2½-in. pipes with no end contraction, the lower end of the pipes projecting from 3 to 4 in. below the lower end of the pile. After the pile enters the ground, the water pressure is maintained at from 80 to 120 lb. A duplex pump, 10 in. in diameter and with an 8-in. stroke, is used, the boiler pressure being from 100 to 120 lb. Between the pump and the pile the water is carried by two lines of 4½-in. pipe and fire hose. Piles with points not less than 10 in. in diameter have been driven to a penetration of 30 ft. in from 10 to 15 min., including the time required for securing the grade, as they are not sawed off.

In numerous contractors' plants the boiler and pump capacity are too small to furnish a large volume of water, and the writer believes that failure to recognize the advantages of the larger plant has been the reason why the jet method has not as yet come into more general use.

The jet method is applicable in any soil which will settle about the pile after the flow of water is stopped. For pure ocean or river sand, it is the best method to adopt.

One great advantage of the jet is that there is absolutely no danger of ruining the pile by over-driving, as shown by the illustrations in Mr. Cole's paper. It is the writer's opinion that piles are very often driven to destruction, under the direction of conscientious but inexperienced inspectors.

As far as the writer is aware, there is no formula for computing the bearing power of piles jetted to place, and, for the bearing capacity, one would have to depend on comparison with driven piles, or on actual tests on piles in similar soils.

The condition of the pile surface, as to roughness or smoothness, cannot have any great effect on the bearing capacity. Small indentures become filled with compressed soil, which, as far as resistance to settlement is concerned, become a portion of the pile; and the frictional resistance to the pile moving through the soil changes to a shear in the soil over the small area in question; the resulting difference in bearing power would be of no practical value.

L. J. MENSCH, M. AM. SOC. C. E. (by letter).—Mr. Cole's paper treats of a subject which deserves serious attention. Very often expensive foundation methods are resorted to where the work could have been done with reinforced concrete piles for less money and in less time. Mr. Mensch.

It is the writer's opinion that Raymond or Simplex piles should only be used for short lengths and where the strata penetrated are well-known from previous test-borings, and consist of easily compressible soils containing very little ground-water. The driving of the steel pile compresses the soil, and there is a tendency to diminish the size of the hole when the pile is withdrawn, which cannot be prevented by the loosely connected No. 20, sheet-metal shell of the Raymond pile, or by the weight of a few feet of concrete, in the case of the Simplex pile. There is much less danger in this respect if the hole is drilled by the ordinary well-driving process and sections of steel pipes are sunk, which are afterward withdrawn, and the hole filled with concrete similar to Simplex piles. The drilling does not compress the soil, and, therefore, there is less tendency to close the hole.

In all made-in-place piles there is considerable danger of obtaining a poor quality of concrete, when dropping it for considerable distances, and a very careful supervision is absolutely necessary in order to obtain reliable results.

When driving Raymond or Simplex piles there exists a danger which has caused failure several times, in that the adjacent green piles are injured by displacement. Ground-water, also, has often caused failures in such piles.

Piles, cast in holes made by the well-driving process, can be used with great advantage where only a few are needed and where it does not pay to install a pile-driving plant, or inside of buildings, or in underpinning buildings, where the driving of piles might injure existing foundations.

Mr. Cole mentions some cases where the steel pipes remained in place. Inasmuch as it is not known how long the pipe may last, it cannot be said that the pile is strengthened by the shell, and, in most cases, the use of such piles represents a waste of money, and a fear on the part of the designer to use reinforced concrete piles moulded in advance.

In France made-in-place piles are often made by the Compressol method. A hole is made in the ground by letting special hammers weighing about 2 tons fall from a height of 50 ft., and then strengthening the wall of the hole by tamping in clay with a hammer of a special shape. Unless great precautions are taken in concreting, the dirt of the wall may be entrained in pouring and a poor pile obtained. The fact that made-in-place piles are often shown to carry considerable

Mr. Mensch. loads under tests can be explained in most cases by the friction of the pile against the upper hard crust which generally overlies the yielding stratum. Tests made by the writer have shown this friction to amount to as much as 1½ tons per sq. ft. of contact surface in good fill. In such cases a shorter pile will do the same work as a longer pile, the greater part of which, being in mud, does not add to the carrying capacity.

In the writer's opinion there are no good reasons why made-in-place piles should be adopted in preference to moulded piles, the latter type being undoubtedly the safer.

Under good management it is possible to build moulded piles for practically the same price as other piles, and in regard to speed of construction, the writer has built piles at the rate of 40 per day and has driven 24 per day, and this is not the limit of speed.

There is no patent on the principle of reinforced concrete piles, most of the patents relating to minor details which are a detriment to, rather than an improvement on, common-sense construction. Concrete piles need only sufficient reinforcement for safe handling before driving. When they are well made and a good driving-cap is used, there is less chance of such a pile breaking than is the case with a timber pile. Concrete piles should have rather a blunt point, or may be cut off square, and, in case the driving is done by a water jet, it is advisable to use a concave point when only one central jet is used. To the writer's knowledge, the longest concrete piles ever driven were 75 ft. long and 16 in. square, reinforced by four 2-in. round bars.

These long piles are very expensive, however, and can be supplanted by a composition pile, such as is used in the wharves and sea walls of San Francisco Harbor. A wooden pile from 50 to 60 ft. long is first driven so that the top projects about 10 ft. above the mud-line, which is from 20 to 30 ft. below the top of the wharf. Over this wooden pile, a hollow concrete pile from 2 to 3 in. thick and having an outside diameter of about 24 in. is driven to a good bearing in the mud. These hollow piles are reinforced by from 4 to 6 sq. in. of steel in a longitudinal direction and by No. 1 wire, 4 in. on centers, in a circumferential direction. The mud inside the pile is ejected by a water jet, and sometimes the water is pumped out, and the interior filled with concrete. The advantage of hollow piles over concrete piles lies in the lower cost of the composition pile and also in the greater stiffness. Besides, the bearing of the 24-in. pile adds considerably to the sustaining power of the composite pile. Such composite piles can also be used with great advantage on land, and are certainly cheaper than long concrete piles.

The writer does not think that Mr. Cole's comparison of a 14 by 14-in. straight pile with a tapered pile 14 in. at the butt and 6 in. at the point, is fair. If he had compared a tapered pile having the

same amount of material as the straight 14 by 14-in. pile, which tapered pile would be 22 in. at the butt and 6 in. at the point, he would have obtained the same surface in both cases, and there is then left in favor of the tapered pile the very considerable wedge action on the ground. Mr. Mensch.

CHARLES H. HIGGINS, ASSOC. M. AM. SOC. C. E. (by letter).— Mr. Higgins. Mr. Cole, in his interesting paper on "Concrete Piles," brings forward a matter of great economic importance.

In his preliminary remarks on wooden piles, he cites instances of such piles exposed by subsequent excavation that were "buckled and crimped into accordion-like folds, and had no sustaining power whatever." Undoubtedly, many wooden piles are injured in driving, particularly when the purpose is to drive to a certain depth without regard to the resistance, as in the case referred to. The author states that:

"In the construction of caisson foundations for sky-scrapers in New York City, it has been the custom in many instances to drive timber piles to form a temporary support for the derrick traveler. These piles remain in place until all material has been excavated."

Probably, if the purpose had been the ordinary one of obtaining resistance to carry a load, driving would have ceased before the piles got into the condition shown, as all who are experienced in such work realize the danger from over-driving; but, even if a pile be damaged in driving, if it is true that the resistance to the last few blows of the hammer measures the carrying capacity of the pile, it can hardly be said that it "has no sustaining power whatever."

Mr. Cole's remarks on timber piles in piers, docks, and similar structures will be accepted by most engineers, but, what is the life of a concrete pile alternately covered and exposed by the waves and tide in this climate? If, under these conditions, the surfaces of some bridge piers and concrete bulkheads deteriorate, the position of a pile or column, not more than 20 in. in diameter, exposed on all sides to the waves and tide, and loaded to several hundred pounds per square inch of section, is one not without hazard.

It is not quite clear to the writer what is meant by "comparable size," on the last line of page 430.* If two piles, one of concrete and the other of wood, have the same diameter at similar places in their length, and the same volume, one would hardly expect the concrete pile to carry double the load of the wooden pile, unless it was heavily reinforced with steel. In such a pile, the steel becomes as great a factor in the cost as the concrete.

In the comparison of the relative cost of concrete and timber pile foundations at Annapolis, Md., cited by the author, the length of the

* *Proceedings*, Am. Soc. C. E., for May, 1909.

Mr. Higgins.

piles is not stated, but the price, $9.50 for wooden piles, in such a large quantity, would indicate a length of not less than 50 ft. As 2 193 wooden piles were to be used, or 855 concrete piles, if it be assumed that 12 tons was the working load for the former, 31 tons would be the load of the latter. Ordinarily, a concrete pile, 50 ft. long and capable of bearing safely 31 tons, would certainly cost not less than $1.20 per lin. ft., or $60 each, and 855 piles at $60 = $51 300, instead of the $17 100 given in the text, or a total of $59 603.20 for the foundation with concrete piles, as against $52 859.18 given as the cost of the foundation with wooden piles. It would be interesting to know what conditions made the estimate quoted possible.

In regard to the made-in-place piles referred to as Class A, the writer has seen such piles driven, and, under certain subsoil conditions, it would seem to be an excellent method; but, remembering how few excavations made in soil which is usually piled do not lay bare unexpected conditions, he would not feel secure in their use.

Taper may be an excellent quality in a pile under certain conditions, but, in the main, the purpose of piling is to transmit the load through a soft stratum to a hard one into which the piles usually do not penetrate far, and, for this purpose, a large area at the point and stiffness as a column would seem to be the chief requisites. The reason wooden piles have a taper is obvious, but the same reason does not obtain when concrete is used.

The behavior of the piles referred to as Type 3, shown on Plate XVI, Figs. 1 and 2, Plate XVII, Fig. 2, and Plate XVIII, Figs. 1 and 2,* was carefully watched by the writer during the making, handling, and driving. They were handled in the manner customary with wooden piles, and required no greater care. The pile shown on Plate XVII, Fig. 2,* after being driven in sand with a hammer weighing 3 500 lb. to an extent that would have crippled a wooden pile, showed no damage on being withdrawn, except for about 2½ ft. of the head, which is shown in the photograph. Aside from the cost, it would be hard to find any objection to them. How they will stand the action of sea water when alternately covered and exposed by the tide and beaten by the waves during freezing weather is a question the writer is trying to determine by observation of one made at the same time as those shown on the plates, and put in position about a year ago. It is still too early to draw any conclusions.

Excellent concrete piles can be made, but their use, except under exceptional conditions, is prohibited by their cost. The reference to a composite wood and concrete pile used at Bridgeport is of interest as it is along this line that the writer believes development will occur. Wooden piles are spliced successfully with a dowel and ferrule. The writer has in mind several important structures carried by such piles,

* *Proceedings*, Am. Soc. C. E., for May, 1909.

where, due to lack of head room, 40-ft. or 50-ft. piles were driven in three or four sections. Therefore, as the chief advantage in the use of concrete piles for foundations occurs when the permanent ground-water level is a number of feet below the surface, as pointed out by Mr. Cole, why cannot a section of cast concrete pile, properly reinforced, be spliced to a wooden pile, thus gaining the advantage of permanency in the concrete above water, and, at the same time, reducing the total cost of the pile by using wood below water where it is equally permanent? Mr. Higgins.

In the case of a 50-ft. pile, where the water level is 10 ft. below the surface, the cost of a concrete pile at $1.20 per lin. ft. = $60, the cost of a composite pile would be:

Wood, 40 lin. ft. at 20 cents per lin. ft.	= $8.00
Concrete, 12 lin. ft. at $1.20 per lin. ft.	= 14.40
Dowel and ferrule	= 5.00
	$27.40

or a saving of $32.61 per pile, over an all-concrete pile, and a saving in excavation and concrete piers over wooden piles cut off at water level.

MAXWELL M. UPSON, ASSOC. M. AM. SOC. C. E.—It is to be regretted that more definite data on the carrying capacity of piles of various shapes and types are not available. The speaker has examined all the available literature on this type of foundation construction and has found comparatively little that can be used as a direct guide in predetermining results. Mr. Upson.

The chief reason for this is that the solution of a piling problem depends entirely on the nature of the soil, and, as all engineers know, there are almost as many kinds of soil and combinations of strata as there are piling jobs. For that reason the engineer who has had the most experience in the design and construction of pile foundations is most loath to place himself in writing on its theories and laws. Because of this lack of information, the speaker has been accumulating data for the past two years, with the object of eventually putting it into a paper to be read before this Society. These data are not yet complete, and it is only with a view of calling attention to information which seems to be at variance with statements in Mr. Cole's paper that the following is submitted.

The speaker desires to refer particularly to the relative advantages of tapered and straight piles. During the past two years it has been his privilege to have charge of the driving of a great many thousand concrete piles, comprising more than 75% of those driven in the United States. Within that time, soil conditions have been encountered in about a hundred different locations. On this information, together

Mr. Upson. with carefully kept records of pile-driving for the last ten years, the following conclusions are based.

The speaker has not attempted to theorize on this information, or to draw up any theoretical tables to look well on paper and be used as an aid to enthusiastic but not always accurate salesmen. He admits that, during his early connection with work of this kind, he made attempts to do so, but found, however, that figures of this character were not borne out in practice, and that every rule seemed to have an infinite number of exceptions.

Mr. Cole seems to be under a misapprehension in regard to the "Type B" pile, as he has classified it only as a "tapered pile." It is not essentially a tapered pile, and, in fact, in several instances, where it seemed advantageous to use a straight or very slightly tapered pile, piles of this shape have been used successfully.

The inventor of this system (prior to its development) spent about 20 years as a wood-piling contractor, and built many of the railroad bridges in Kansas and Nebraska. It was his observation that tapering wooden piles in friction soil gave the better carrying capacity. On that account, when he invented the "Type B" system, known as the "Raymond," he experimented with a variety of tapers. His original design made it essential that the piles should have some taper in order to keep the shell from creeping up on the core. Since that time certain changes and variations have been evolved which obviate the necessity of a tapering form and make it possible to use a perfectly straight core. The company which puts in this "Type B" pile is prepared to put in either straight or tapered piles.

The fact that 90% of the work executed by this company is based on tapered piles, indicates its firm conviction that this particular form has advantage in carrying capacity. This conviction is arrived at by information received from two sources: First, from a careful record of the resistance encountered in driving; and second, from a careful series of loading tests.

The method of placing these piles consists of driving, with a steam hammer, a steel core encased in a shell, which makes it possible to observe the penetration under each blow of the hammer; and since the weight of the core and hammer is constant, a comparison of the carrying capacity of each pile can be predetermined very accurately by noting the final resistance to each blow of the hammer, or, in pile-driving terms, the number of blows to the last inch. As a result of a large number of tests, it has been found that this method of measuring the resistance is practically as good as weighting the pile.

Loading tests have been carried on in which the real carrying power of the pile has been checked with the resistance demonstrated at the time of driving.

The following are the results of tests made in Boston in the fall

of 1906, on three piles having a variety of tapers: These tests were conducted under the supervision of a prominent engineer who contended that, for the soil in question, a straight pile had advantages over a tapered pile, taking an attitude similar to that expressed by Mr. Cole, viz., that increased surface area afforded increased friction and consequently increased carrying capacity. To make sure of this point, two piles each 20 ft. long, were driven within a few feet of one another. One core, known as "Core *C*," was 13 in. at the point and 18 in. at the butt, and the other, "Core *A*," was 6 in. at the point and 20 in. at the butt. It should be observed that it required 944 blows of

Mr. Upson.

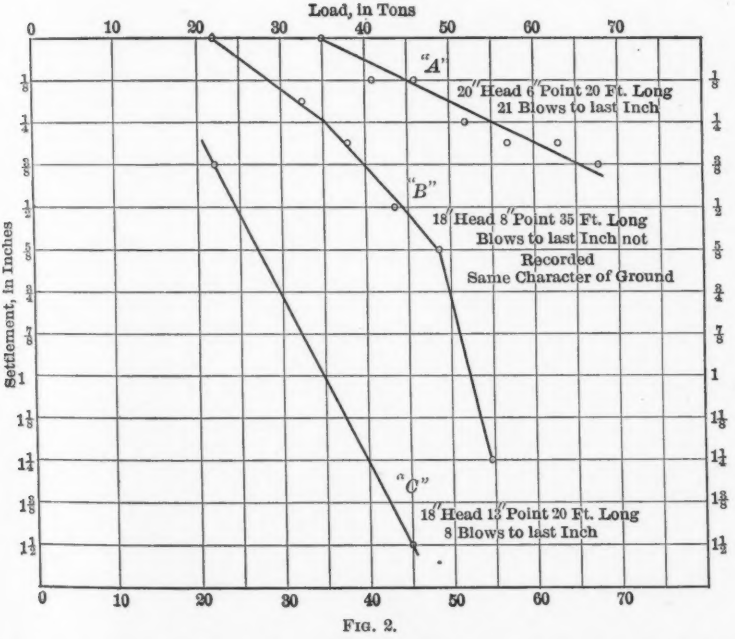

FIG. 2.

a steam hammer to drive Core *C* 20 ft., the last inch of penetration requiring 8 blows. Core *A*, with the increased taper, required 875 blows to drive it 20 ft. into the ground, the last inch of penetration requiring 21 blows.

At the expiration of about one month, both these piles were carefully tested, the result of which is shown in Fig. 2. Pile *C*, with the slight taper, showed an initial settlement of $\frac{3}{8}$ in. with a load of 22 tons, while Pile *A* did not show a settlement of $\frac{3}{8}$ in. until after 63 tons had been placed upon it; in fact, it showed no settlement whatever under a load of 35 tons, and an unappreciable settlement up to 63

Mr. Upson. tons. These results are particularly interesting in view of the fact that the engineer who made the tests did so for the specific purpose of demonstrating that a straight pile has increased carrying capacity. The results of these tests have been demonstrated again and again in the experience of the company with which the speaker is connected.

It is his firm conviction that the success of the work undertaken by this company, with an experience of eight years, in which no settlements have ever been encountered, and covering much more than half of the concrete piles driven in the United States, is due, first, to the absolute test which is afforded by watching the penetration of the pile while being driven; second, to the assurance that each pile is perfect, because the shell permits careful inspection; and third, because of the taper of the pile, which, in the speaker's opinion, increases the friction in proportion to the load imposed.

It is not contended that there are not many places in which a straight pile has its advantages. As a matter of fact, it is necessary where the top soil has little or no bearing capacity or may be washed away, and where the underlying strata are either hardpan, gravel, or rock. It is surprising, however, how infrequently these conditions are met in piling propositions.

It does not seem wise at this time to attempt to enter into a discussion of the theory of why a tapered pile has a greater carrying capacity than a straight pile. There are several theories, and the speaker believes that they are obvious to most deep-thinking engineers. At a later date he hopes to present his views on this subject to the Society.

As a matter of economic interest, attention is called to the record (on Fig. 2) of the test of a 35-ft. pile driven in the same ground; this pile measured 8 in. at the point and 18 in. at the butt. It will be observed that on account of its decreased taper it was possible to secure a penetration some 15 ft. more than with the 20-ft. pile having the maximum taper; but, notwithstanding the increased length, its carrying capacity under a load was not as great as that of the short pile.

The failure to recognize this feature in piling has caused a material increase in the cost of many concrete pile foundations, for the reason that contracts are usually awarded on the basis of the lowest price per linear foot. It is well-known that, in a measure, the cost of piling is inversely proportional to the number of feet involved in the contract. It is obvious, therefore, that the bidder who contemplates the use of a straight or nearly straight pile or form realizes, unless the site overlies hardpan or rock, that any desired penetration may be secured. In other words, a straight pile, 40 ft. long, can be used where a tapered pile, 20 ft. long, would have been more effective. This means that twice the necessary linear feet of piling has to be paid for. This prin-

ciple is well illustrated in New Orleans, where some recent important structures are resting on 20-ft. tapered concrete piles which were substituted for 70-ft. wooden piles. Tests on the short tapered concrete piles showed an increased carrying capacity of from 25 to 50% over the 70-ft. wooden piles. Mr. Upson.

The relationship between the number of blows per inch and the carrying capacity under test is well illustrated in the curves on Fig. 2. It will be observed that the pile with the slight taper brought up at a resistance of 8 blows per inch, while the pile with the increased taper required 20 blows per inch. The first pile showed a settlement of $\frac{3}{8}$ in. under 22 tons and the tapered pile a settlement of $\frac{3}{8}$ in. under 65 tons. The ratio between the carrying capacity and the number of blows per inch practically agreed.

It is to be regretted that the method of checking the carrying capacity of piles by their penetration cannot be worked out in a built-up pile. This is due to two reasons: First, their mass is variable, and is so great that it is difficult to secure uniform conditions; and second, in order to preserve the pile, an elaborate system of cushions has to be used, the efficiency of which may vary almost 100 per cent. In other words, when a cushion block is new and in good working order, it may transmit 75% of the blow to the pile, and when it is old or out of order it may absorb practically all the blow. This is a feature which is sometimes not considered by engineers who keep a record of the driving of built-up piles.

It has been well brought out by Mr. Cole that there is serious objection to the use of a permanent shell, on account of its cost. With this in mind, the speaker has made a series of investigations and experiments with a view of eliminating this from the "made-in-place type" of pile. As yet it must be admitted that a satisfactory way of obviating this expense has not been found. The more the matter is studied, the more difficult it seems. Piles must be driven in succession, within a comparatively short distance of one another, and for this reason it is found that, unless a shell or protection of some kind remains around the soft concrete, the driving of adjacent piles compresses the soil and cuts off or seriously warps and displaces the green piles previously driven.

Thomas C. Desmond, Esq. (by letter.)—The conclusion of Mr. Cole's paper contains a comparison of the relative supporting power of tapered piles and piles of uniform diameter. The writer respectfully takes exception to this comparison for two reasons: First, Mr. Cole does not compare a pile of uniform diameter with a tapered pile of equal volume; and second, in his computation of "skin-friction" area, he does not take into account the wedge action of the tapered pile, which is a very important factor in its supporting power. Mr. Desmond.

In a thesis on concrete piles, offered for the degree of Bachelor of

Mr. Desmond. Science at the Massachusetts Institute of Technology, in June, 1909, the writer made an application of the Rankine theory of earth pressure to a discussion of the relative supporting power of tapered piles and of piles of uniform diameter. As far as he knows, no computations have ever before been published, wherein the reactions of the surrounding earth, which go to make up the supporting power of a pile, have been computed according to any theory of earth pressure. It is respectfully hoped, therefore, that the following computations, which are based on Rankine's theory of earth pressure, may be of interest to the Society.

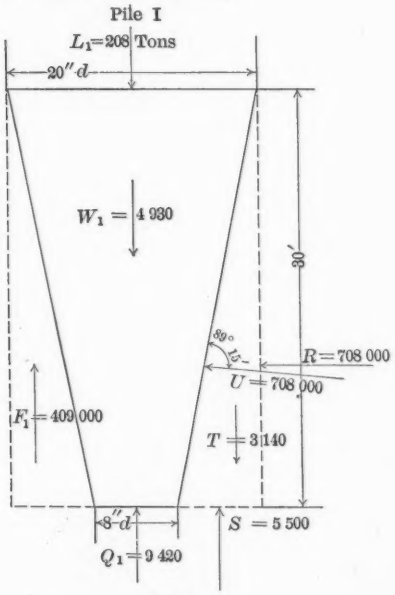

FIG. 3.

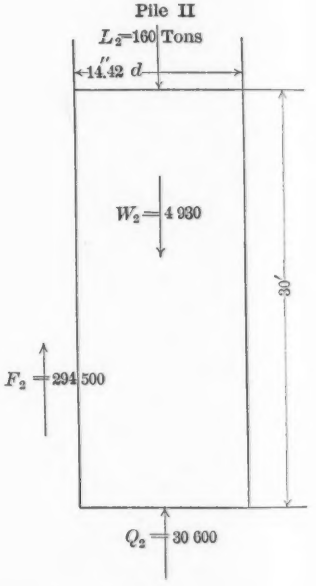

FIG. 4.

Two piles will be considered. The first, shown in Fig. 3, is 30 ft. long, with a diameter of 20 in. at the butt, tapering to 8 in. at the point. These are the dimensions of a standard, 30-ft. Raymond pile. The second pile, shown in Fig. 4, is 30 ft. long, with a uniform diameter of 14.42 in. This diameter is choosen in order that the second pile may contain the same volume of concrete as the first.

Computations have been made for the theoretical ultimate supporting power of the two piles, assuming values of B = the angle of repose of earth, ranging from 0° to 30°. The results are plotted in the curves of Fig. 5, where the abscissas represent the values of B, and the ordinates the supporting power, in tons. The soil is assumed to be homogeneous from the butt to the point of the pile.

In order that the method of making these computations and the assumptions which underlie them may be clearly understood, the computations for the case of $B = 30°$ will be given in full. Mr. Desmond.

Let B = the angle of repose of earth on earth, assumed = 30°,
B_1 = the angle of friction of earth on concrete, assumed = B = 30°,
W = the weight of a cubic foot of earth, assumed = 100 lb.,
G = the weight of a cubic foot of concrete, assumed = 145 lb.,
A_1 = the area of the butt of Pile I = $0.7854 \times 400 = 314.16$ sq. in.,
A_2 = the area of the point of Pile I = $0.7854 \times 64 = 50.27$ sq. in.

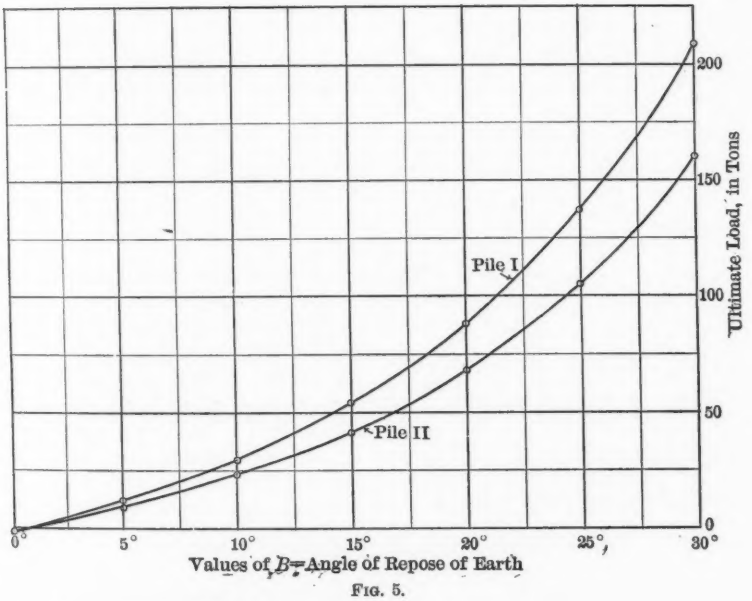

FIG. 5.

Then the volume of Pile I

$$= 120\,(314.16 + 50.27 + \sqrt{314.16 \times 50.27})$$
$$= 120\,(364.43 + \sqrt{15\,800})$$
$$= 120\,(364.43 + 125.7)$$
$$= 120 \times 490.1 = 58\,800 \text{ cu. in.}$$

Therefore, in order that Pile II may have the same volume as Pile I, the cross-sectional area of Pile II must be $= \frac{490.1}{3} = 163.4$ sq. in.

Mr. Desmond.

Let d_2 = the diameter of Pile II,

then $d^2_2 = \dfrac{163.4}{0.7854} = 208.$

Hence, $d_2 = 14.42$ in.

$$\text{Weight of Pile I} = \text{weight of Pile II} = \frac{58\,300 \times 145}{1\,728} = 4\,930 \text{ lb.}$$

Assuming the surface of the ground to be a horizontal plane, and calling P = maximum intensity of pressure, in pounds per square foot, possible on a horizontal plane at a depth, d, below the surface, it is shown in the demonstrations of the Rankine theory of earth pressure that

$$P = w\,d\left(\frac{1 + \sin.\,B}{1 - \sin.\,B}\right)^2$$

That is, in this case when $B = 30°$

$$P = 100 \times 30\left(\frac{1.5}{0.5}\right)^2 = 37\,000 \text{ lb. per sq. ft.}$$

Therefore, the supporting force, Q, acting on the point of Pile I $= 27\,000 \times \dfrac{50.27}{144} = 9\,420$ lb. Also, Q_2 acting on the point of Pile II $= 27\,000 \times \dfrac{163.4}{144} = 30\,600$ lb.

Again, it is shown in the demonstrations of the Rankine theory of earth pressure that, when the surface of the ground is a plane, the pressure on any vertical plane is parallel to the surface, and the intensity of pressure on any plane varies directly with the depth. Also, when the surface of the ground is horizontal, letting q = maximum intensity of pressure possible on a vertical plane, in pounds per square foot, then $q = w\,d\,\dfrac{1 + \sin.\,B}{1 - \sin.\,B}$

That is, for this case, when $B = 30°$,

$$q = 100 \times 30 \times \frac{1.5}{0.5} = 9\,000 \text{ lb. per sq. ft.}$$

Considering Pile II, since the intensity of pressure is zero at the butt of the pile and 9 000 lb. per sq. ft. at the point, and the intensity varies uniformly from butt to point, then the total perpendicular pressure on the lateral surface of the pile

$$= \frac{4\,500 \times 3.1416 \times 14.42}{144} \times 360 = 510\,000 \text{ lb.}$$

Hence, F_2 = the supporting power of Pile II due to friction
$= 510\,000 \times \tan. B_1 = 510\,000 \times 0.5774 = 294\,500$ lb.

Therefore, the ultimate supporting power of Pile II
$= 294\,500 + 30\,600 - 4\,930 = 320\,000$ lb. $= 160$ tons.

Mr. Desmond.

Let an imaginary cylinder be considered, the projection of which on a horizontal plane is the butt of Pile I, and the outlines of which are indicated by the dotted lines in Fig. 3. Then the total pressure on the lateral surface of Pile I is the resultant of three forces, namely (1), the total pressure on the lateral surface of the imaginary cylinder, acting horizontally $= R$; (2), the total pressure on the area of the base of the imaginary cylinder less the area of the point of the pile, acting vertically upward $= S$; and (3), the weight of a volume of earth equal to the volume of the cylinder less the volume of the pile, acting vertically downward $= T$.

$$\text{The value of } R = 4\,500 \times \frac{3.1416 \times 20 \times 360}{144} = 708\,000 \text{ lb.}$$

Also, assuming that the intensity of pressure on that part of the base of the imaginary cylinder which is not included in the area of the point of the pile is merely the ordinary pressure due to the depth below the surface of the ground $= w\,d$, according to the Rankine theory, then

$$S = 100 \times 30 \times \frac{(314.16 - 50.27)}{144}$$

$$S = 3\,000 \times \frac{263.89}{144} = 5\,500 \text{ lb.}$$

$$\text{Also, } T = \frac{(360 \times 314.16 - 58\,800)}{1\,728} \times 100$$

$$T = \frac{54\,200}{1\,728} \times 100 = 3\,140 \text{ lb.}$$

Hence the vertical component of the total pressure on the lateral surface of Pile I $= S - T = 2\,360$ lb., and the horizontal component $= R = 708\,000$ lb. Within the limits of slide-rule accuracy, the resultant total pressure on the lateral surface of the pile is therefore also equal to $R = 708\,000$ lb., and the angle which its direction makes with a horizontal plane

$$= \text{tan.}^{-1} = \frac{2\,360}{708\,000} = 0.00334 = 0° 12'.$$

But the lateral surface of the pile is at an angle with a vertical plane $= \text{tan.}^{-1} = \frac{6}{360} = 0.01667 = 0° 57'$.

Hence $U =$ the resultant total pressure, makes an angle

$$= 90° - 0° 57' + 0° 12' = 89° 15'$$

with the lateral surface of the pile, as shown in Fig. 3.

The component of this pressure normal to the surface of the pile

$= 708\,000 \times \text{sin. } 89° 15' = 708\,000 \times 0.9991.$

$= 708\,000$ lb., approximately.

Hence the total frictional resistance of the pile

$= 708\,000 \times \text{tan. } B_1 = 708\,000 \times 0.57735 = 409\,000$ lb.

Mr. Desmond.

The vertical component of this frictional resistance $= F_1$
$= 409\,000 \times \cos. 0° 57' = 409\,000 \times 0.99986 = 409\,000$ lb., approximately.

Hence the total supporting power of Pile I
$= 409\,000 + 5\,500 - 3\,140 + 9\,420 - 4\,930$
$= 416\,000$ lb. $= 208$ tons.

The results of the computations, which are plotted on Fig. 5, indicate the superiority of tapered piles as regards theoretical supporting power. It must be carefully noted, however, that these computations are for soils which are homogeneous from the butt to the point of the pile. In the case so often met in practice, where the piles are driven through soil with a low angle of repose at the butt of the pile and a high angle of repose at the point, that is, where the piles act practically as columns unrelieved of their load until they reach a solid substratum, then, obviously, piles of uniform or increasing diameter would be preferable.

MEMOIRS OF DECEASED MEMBERS.

NOTE.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

CABELL BRECKINRIDGE, M. Am. Soc. C. E.*

DIED NOVEMBER 13TH, 1907.

Cabell Breckinridge was born in Louisville, Kentucky, on November 22d, 1846. He was the youngest son of the Rev. Dr. William Lewis Breckinridge and his wife, Frances Caroline Prevost. His father was for nearly twenty-five years pastor of the First Presbyterian Church of Louisville, and later was at different times President of Oakland College, Mississippi, and of Centre College, Danville, Kentucky. His paternal grandfather was John Breckinridge of Virginia, Attorney-General of the United States during a portion of President Jefferson's administration. His grandmother was Mary Hopkins Cabell of Virginia. On the maternal side he was descended from John Bartow Prevost, a Judge of the Supreme Court of Louisiana, and his wife, Frances Ann Smith, the daughter of Dr. S. S. Smith, President of Princeton College, and granddaughter of John Witherspoon, President of Princeton College and a signer of the Declaration of Independence.

During his childhood, Cabell Breckinridge attended the school of the Rev. Joseph J. Bullock, at Walnut Hill, near Lexington, Kentucky, and was afterward a student at Oakland College, Mississippi, and at Centre College, Danville, Kentucky. At the age of eighteen, he left college before graduation, and joined an engineering party which, under the direction of Colonel Brodhead, was engaged on a reconnaissance for the Louisville and Nashville Railroad in Rockcastle County, Kentucky. From this time until the end of his life, Mr. Breckinridge was engaged, with the exception of a few interruptions in his later years on account of sickness, in the practice of his profession as a civil engineer. The record of his engagements, summarized from a memorandum left by him, is as follows:

1865—Rodman on Louisville extension of the Louisville and Nashville Railroad.

1866-70—Leveler, Transitman, and Resident Engineer on the Louisville, Cincinnati, and Lexington, now a part of the Louisville and Nashville Railroad.

1870—Engaged on a line in Wisconsin, afterward the Wisconsin Central Railway.

1871-73—Resident Engineer and Assistant Division Engineer on the Chesapeake and Ohio Railway.

* Memoir prepared by Onward Bates, President, Am. Soc. C. E.

1874-78—Division Engineer and Expert on the Cincinnati Southern Railway.
1878-80—United States Government service: Assistant Engineer on Cascade Locks.
1880-83—Chief Engineer, Alabama, Great Southern Railway, now a portion of the Cincinnati, New Orleans and Texas Pacific Railway System.
1883-84—Division Engineer on the Philadelphia Branch of the Baltimore and Ohio Railroad.
1884—United States Government service: Assistant Engineer on the Red and White Rivers.
1885-86—Engaged with the Beaver Creek Coal Company.
1886—Reconnaissance for Kentucky Central Railway (Louisville and Nashville) for lines to coal fields, and survey for a bridge over the Ohio River at Cincinnati.
1887—Division Engineer, Tennessee Midland Railroad, making surveys.
1888—With Cincinnati Southern and Louisville and Nashville Railways.
1889—United States Government: Ice Harbor Lock, Marietta, Ohio.
1890-91—Division Engineer, Kentucky Central Railroad, making surveys.
1892-94—Engaged on asphalt paving for the City of Covington, Ky.
1895-96—Division Engineer, Tennessee Central Railroad.
1896—State of Tennessee: Brushy Mountain Coal Mines.
1896-97—With Louisville and Nashville Railway, inspecting masonry, rebuilding bridges, and putting in retaining walls at Benson washout.
1897—On the United States Government survey of Warrior River.
1898-1900—Engaged on surveys for coal roads.
1901—Division Engineer, Texas and Pacific Railway, on new lines.
1902—Division Engineer on Seaboard Air Line, on survey from Rockmart to Atlanta.
1903-04—Chief Engineer, Tennessee Railway, and Tennessee Development Company.
1904-06—In private practice.
1907—Engaged on electric lines.

From this record it will be observed that Mr. Breckinridge's professional labors extended over a period of more than forty years. In that time he contributed his full share in the work of developing the prosperity of this Nation during the era of its greatest advancement. In the sense of giving his life's service to the country for the improvement of its physical condition, and the building of its transportation systems, he was a patriot and good citizen. His name deserves an honored place among those civil engineers to whom the Nation owes a debt of gratitude, for lives spent in the accomplishment of the common good, without regard for personal rewards. The nature of Mr. Breckinridge's engagements, and his devotion to the work immediately in hand, stood in the way of a widely distributed acquaintance and the general reputation he would have enjoyed under other conditions. He

was greatly esteemed as an engineer by his associates who were capable of judging his attainments, and their regard for his personal qualities was remarkable. The writer of this memoir remembers him in 1869, as a splendid specimen of young manhood, with every prospect of professional success. That his ultimate success may have fallen short of what was expected by his friends, may be attributed to a character which was always kind and just to others and yet failed to do justice to himself. Any criticism of that character is lost in the feeling of affectionate regard entertained for him by those who enjoyed his acquaintance. Although the writer met Mr. Breckinridge early in his career, he was never placed in intimate relations with him, and in preparing this memoir has obtained the data for it from those who were associated with Mr. Breckinridge in his professional life. Such of his associates as are now living, including those under whose direction he was employed, and those working directly under him, as well as others who had the opportunity of judging him as an Engineer and Man, have been consulted by correspondence. While all these correspondents agree in a high estimate of him as an Engineer, they seem moved to lay a greater stress upon his manly and lovable qualities. It is proper to mention these tributes to his character in a sketch of his life, and this cannot be done in a better manner than by quoting the exact words of those who were best acquainted with him:

"He was an intelligent, energetic and capable Engineer, ready for service night or day. * * * I was very fond of him, considering him an exceptionally capable and agreeable assistant."

"Large, both in mind and body, his was a most lovable nature. He was a big-hearted, brave, generous fellow, and always a courteous gentleman."

"He was a man of high integrity, a genial companion, and true friend. He was an engineer of ability and good judgment in handling work."

"His wonderful grasp of details, and minutiæ of the profession, astounded me. * * * Personally, he was a most delightful companion, and always courteous, even under extreme provocation, so much so as to completely control an engineering party by this method alone."

"Regarding his personal character, all of us who knew him can speak in the highest terms. He was a typical Southern gentleman, of the highest, educated class. The very soul of personal honor, he was notably loyal to, and ever ready to serve his friends. Quick tempered and brave to a fault, but even more just and generous."

"He was a very accomplished engineer, a man of high capacity, absolute probity, a fine sense of honor, and great devotion to duty. * * * He was courteous and hospitable to the highest degree. In his home life he was charming. I have seen him gather his wife, ill or weary, in his arms and carry her up-stairs. He was, I may add, one of the very finest looking men, face and figure, I ever met, and a fine figure on horseback."

"I served with and under Cabell Breckinridge during the construction of the New River Division of the Chesapeake and Ohio Railroad, 1870-73, and of the Cincinnati Southern Railway in 1875-78. He had the confidence and esteem of his superiors, and the respect of his associates and subordinates. He was a man of great energy and executive ability, with the highest ideals of courtesy, honor and integrity."

"All of our people knew him and loved him. The men who worked under him had the greatest confidence in him as an engineer and loved him as a man. From a professional point of view he was in the front rank. As a man, he possessed those qualities which endeared him to every one he met. He was brave, he was gentle, and trusted his friends implicitly. He was possessed of the highest honor."

"His work has always stood bearing testimony to his ability and faithfulness as an engineer. As to his personality, I have never known a more generous, loving and lovable nature; true and ever loyal to his friends; kind and courteous and thoughtful for strangers and those who worked for him. He made friends wherever his work called him. With a tender love of the beautiful, both in Nature and Art, he was in thorough sympathy with the artistic temperament of his gifted wife."

"He was an engineer of no inconsiderable accomplishments, an excellent organizer, an indefatigable worker, and a man of much force and vigor. As a man and gentleman, he had but few peers, and no superiors. A man of distinguished appearance, of fine presence, of courtly manners and of natural dignity, he was as much respected for the high class and order of his characteristics as he was beloved for the tenderness, gentleness, and sincerity of his nature. No man for whom I have ever worked commanded more of my respect or gained a stronger hold upon my personal affections."

If the estimate of Mr. Breckinridge's character, which may be formed from the testimony of his friends as herein stated, had been given in the words of his biographer, the latter might have been suspected of using his office to eulogize a friend, and he is grateful that he is able to use the words of others who were in a position to speak with knowledge.

Mr. Breckinridge was happily married to Miss Julia Marshall, of a distinguished Southern family. He is survived by his widow, a daughter and a son, the latter a banker in New Orleans.

Cabell Breckinridge was elected a Member of the American Society of Civil Engineers on June 1st, 1881. He died at the age of sixty-one at the Presbyterian Hospital in Atlanta, Georgia, of Bright's disease, and his body was interred at Frankfort, Kentucky.

Vol. XXXV. OCTOBER, 1909. No. 8.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

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THE NEW YORK TUNNEL EXTENSION OF THE PENNSYLVANIA RAILROAD. THE CROSS-TOWN TUNNELS.

By James H. Brace and Francis Mason, Members, Am. Soc. C. E.

To be Presented December 1st, 1909.

In this paper, it is proposed to describe the construction of the tunnels extending eastward from the easterly extension of the Terminal Station to the permanent shafts east of First Avenue.

They were located under 32d and 33d Streets from the station to Second Avenue, and thence, curving to the left, passed under private property and First Avenue to the shafts, as described in a preceding paper. Typical cross-sections of the tunnels are shown on Plate LXIV.*

On May 29th, 1905, a contract was entered into with the United Engineering and Contracting Company for the performance of this work. This contract provided that work on each pair of tunnels should be carried on from two shafts. The first, here referred to as the First Avenue Shafts, were located just east of that avenue and directly over the line of the tunnels; the other two, called the Intermediate Shafts, were located on private property to the north of

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

* *Proceedings*, Am. Soc. C. E., for September, 1909.

each pair of tunnels in the blocks between Fourth and Madison Avenues. It was originally intended to do all the work of construction from these four shafts. Workings were started both east and west from the Intermediate Shafts, and those to the west were to be continued to the Terminal Station. After the change of plans, described in a previous paper, it was decided to sink a third shaft on each line. These were known as the West Shafts, and were located between Sixth and Seventh Avenues. Finally, it was found necessary to build a portion of the tunnels on each line west of Sixth Avenue in open cut. The locations of the shafts are shown on Plate LXVI.*

The First Avenue shafts were built by S. Pearson and Son, Inc., for the joint use of the two contractors, as described in the paper on the tunnels under the East River. While the shafts were being sunk, the full-sized tunnels were excavated westward by the contractor for the river tunnels for a distance of 50 ft., and top headings for 50 ft. farther. By this means, injury to the caissons and to the contractor's plant in the shafts by the subsequent work in the Cross-Town Tunnels was avoided. The west half of the shaft was for the exclusive use of the contractor for the Cross-Town Tunnels.

Contractor's Plant.

The method of handling the work adopted by the contractor was, broadly speaking, as follows: Excavation was usually carried on by modifications of the top-heading and bench method, the bench being carried as close to the face as possible in order to allow the muck from the heading to be blasted over the bench into the full section. The spoil was loaded into 3-yd. buckets (designed by the contractor and hereinafter described), by steam shovels operated by compressed air, and hauled to the shafts by electric locomotives. Electrically-operated telphers, suspended from a timber trestle, hoisted the buckets, and, traveling on a mono-rail track, deposited them on wagons for transportation to the dock. Arriving at the dock, the buckets were lifted by electrically-operated stiff-leg derricks and their contents deposited on scows for final disposal. The spoil was thus transported from the heading to the scow without breaking bulk.

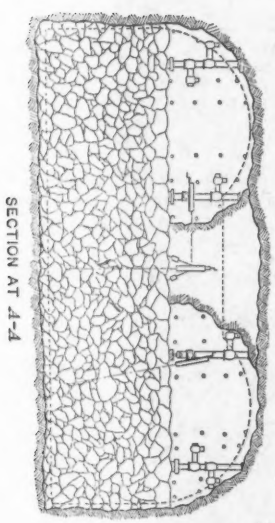
When concreting was in progress, the spoil buckets were returned to the shafts loaded with sand and stone. The concrete materials were

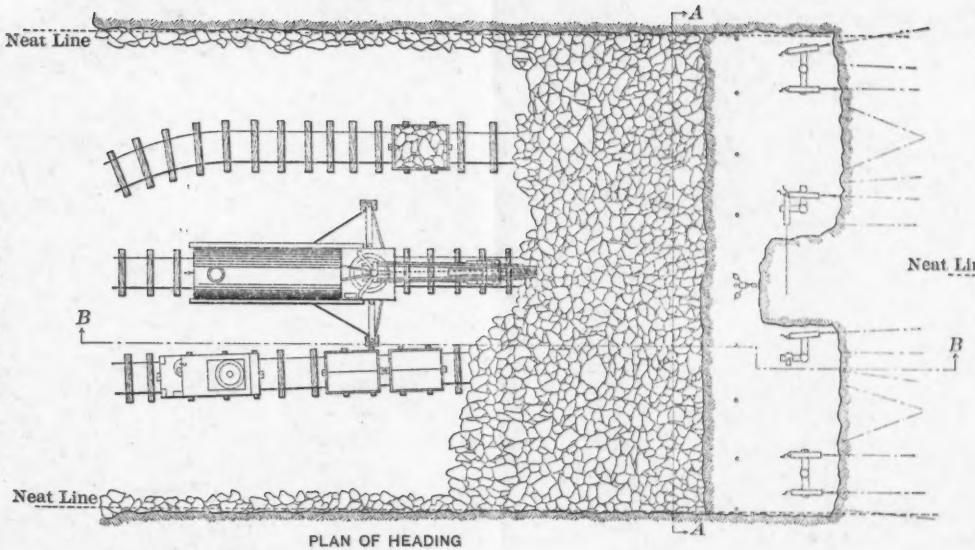
* *Proceedings*, Am. Soc. C. E., for September, 1909.

deposited in storage bins placed in the shafts, from which they were fed to the mixers located at the foot of the shaft about on a level with the crown of the tunnels. The concrete was transported to the forms in side-dump, steel, concrete cars, hauled by the electric locomotives.

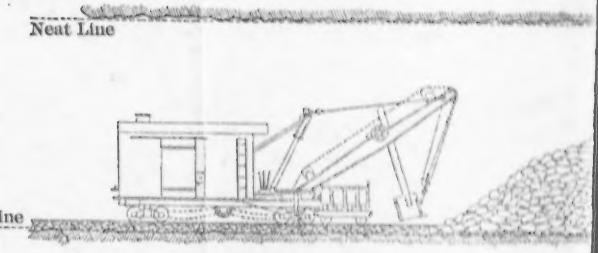
Electrical power was adopted largely on account of the restricted area at the shaft sites, where a steam plant would have occupied considerable space of great value for other purposes. The installation of a steam plant at the Intermediate Shafts, which were located in a high-class residential district, would have been highly objectionable to the neighboring property owners, on account of the attendant noise, smoke, and dirt, and, in addition, the cost of the transportation of fuel would have been a serious burden. Except for the forges and, toward the last, the steam locomotives, not a pound of coal was burned on the work. The use of the bucket and telpher also eliminated most of the objectionable noise incident to the transfer of spoil from tunnel cars to ordinary wagons at the shaft sites. Power plants were installed at the North Shaft near First Avenue and at the rear of the 33d Street Intermediate Shaft.

First Avenue Plant.—Fig. 1, Plate LXXVIII, is a general view of the First Avenue plant. The power-house at the corner of 34th Street and First Avenue supplied compressed air for operating drills, shovels, pumps, and hoists in the tunnels driven from the river shafts, and in it three Laidlaw-Dunn-Gordon compressors were installed. The largest was a 32 by 20 by 30-in., two-stage, cross-compound, direct-connected to a Fort Wayne 480-h.p., 230-volt, direct-current, constant-speed motor run at 100 rev. per min. This compressor was rated at 2 870 cu. ft. of free air per minute at a pressure of 100 lb. It was governed by throttling the suction, the governor being controlled by the pressure in the air receiver and the motor running continuously at a constant speed. The two others were of similar type, one was 22½ by 14 by 18-in., rated at 1 250 cu. ft. of free air at a pressure of 100 lb., the other was 16 by 10 by 18-in., rated at 630 cu. ft. They were fitted with 9-ft. fly-wheels, and were driven at 150 rev. per min. by 105-h.p., General Electric, 220-volt, compound-wound, direct-current motors running at 655 rev. per min. The larger of these two compressors was driven by two of the motors belted in tandem, and the smaller was belt-connected to a third motor. The compressors were water-jacketed and had small inter-coolers, the water supply for which



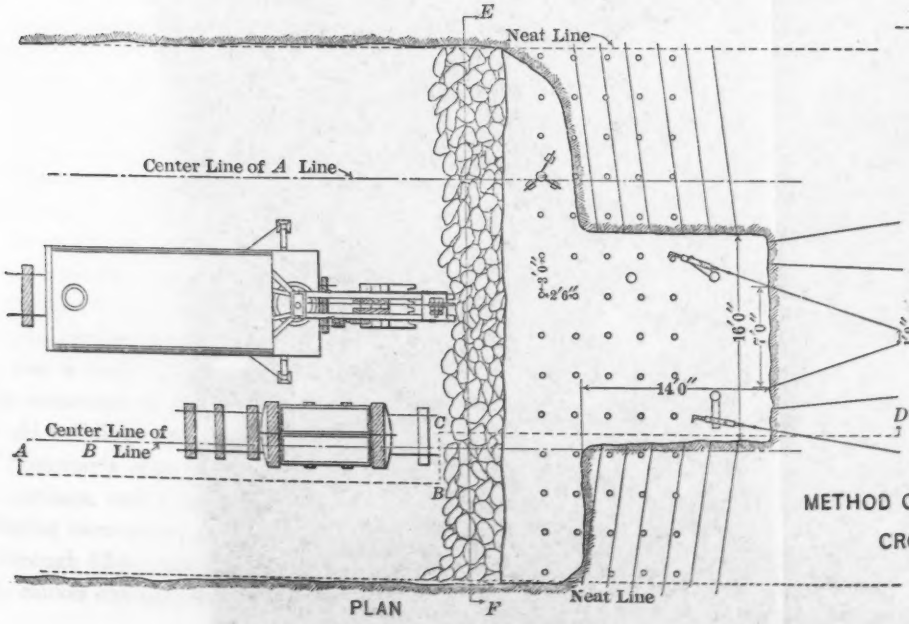


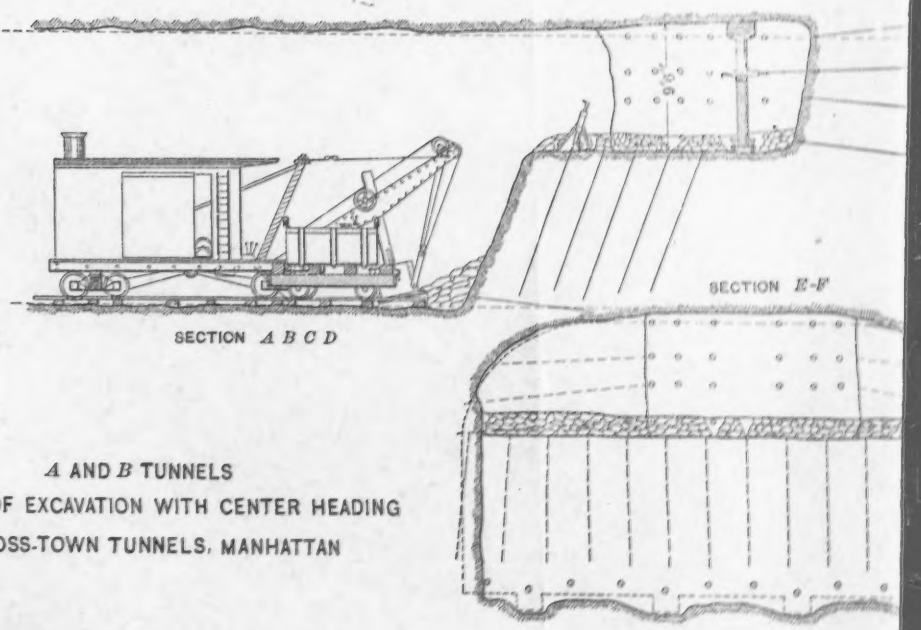
PLAN OF HEADING



LONGITUDINAL SECTION AT *B-B*

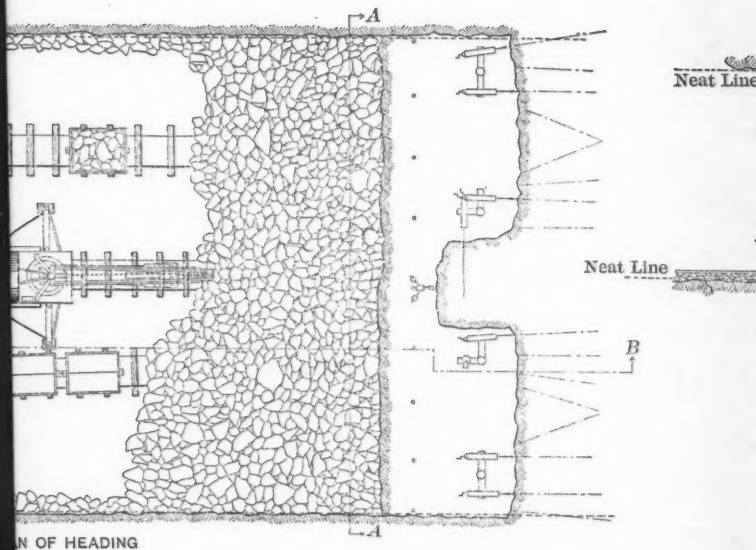
EXCAVATION
WESTWARD FROM END OF ROCK DIVIDING
TWIN TUNNELS *C* AND *D*





SECTION *A B C D*

A AND *B* TUNNELS
METHOD OF EXCAVATION WITH CENTER HEADING
CROSS-TOWN TUNNELS, MANHATTAN



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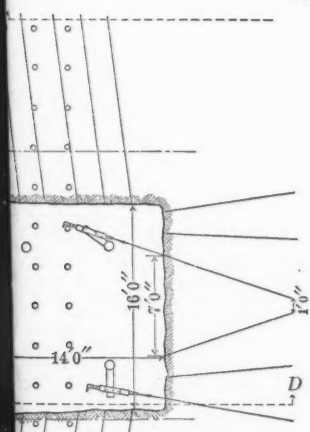
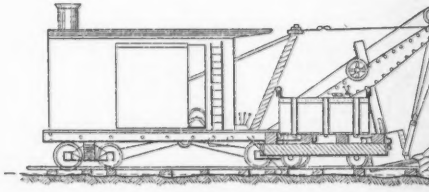


Line

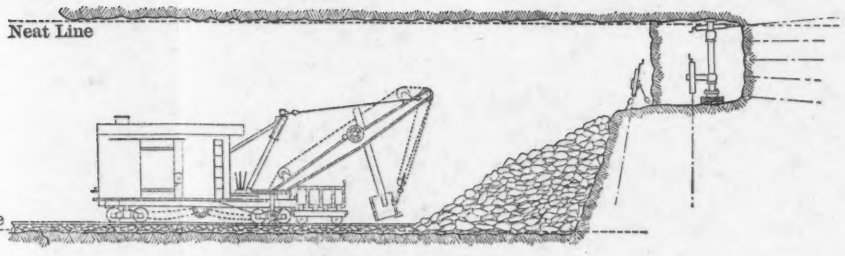


SECTION *A B C D*

A AND *B* TUNNELS

METHOD OF EXCAVATION WITH CENTER HEADING

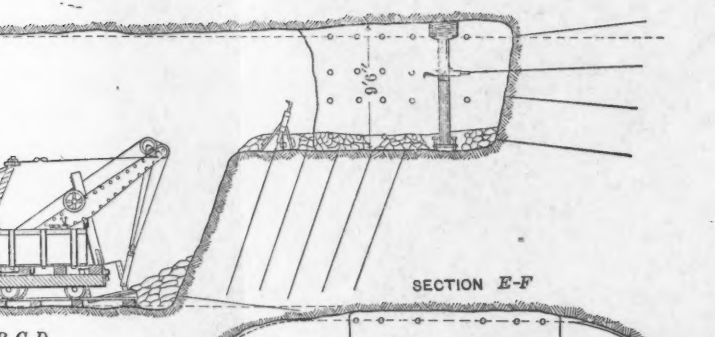
CROSS-TOWN TUNNELS, MANHATTAN



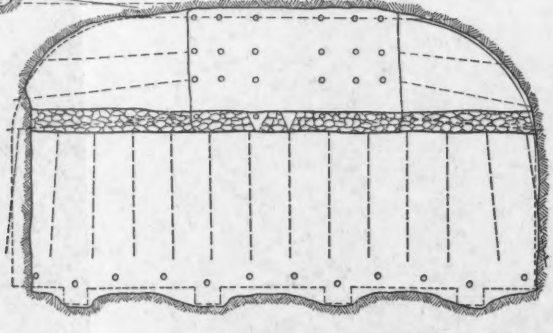
LONGITUDINAL SECTION AT *B-B*

B

EXCAVATION
WESTWARD FROM END OF ROCK DIVIDING WALL[4]
TWIN TUNNELS *C* AND *D*



SECTION *E-F*



B C D

R HEADING
TTAN

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was itself cooled in a Wheeler Condenser and Engineering Company's water-cooling tower. The pump and the blower operating it were electrically driven.

The telphers, used for hoisting muck from the tunnels and for lowering supplies, were each hung from single rails on a timber trestle, about 40 ft. high, spanning and connecting the two shafts. One machine was provided for each shaft, and where their tracks crossed 33d Street they were separated sufficiently to permit the machines to pass each other. At this point, and covering the street, a large platform was provided, on which the trucks were loaded and unloaded (Fig. 2, Plate LXXVIII), and from which they descended by an incline on First Avenue leading south to 32d Street. The platform also covered practically all the yard at the South Shaft and materially increased the available working area. The telphers were built by the Dodge Cold Storage Company, and were operated by a 75-h.p. General Electric motor for hoisting and a 15-h.p. Northern Electric Company motor for propulsion. Their rated lifting capacity was 10 000 lb. at a speed of 200 ft. per min.

The carpenter shop and machine-shop, both of which served the entire work, were conveniently located in small buildings on the loading platform. In the former the saws were each run independently by small electric motors suspended under the platform. The heavy forms and form carriages used in lining the tunnels with concrete were fabricated and stored on the platform outside. The machine-shop lathes, etc., were all belted to one shaft driven by an 8-h.p. General Electric motor. Above the machine-shop was a locker-room and below it on the street level was the main blacksmith shop for the work. Subsidiary blacksmith shops were located at each of the other shafts. The storeroom and additional locker-rooms were located above the power-plant in the North Shaft yard, and isolated from the other structures was a small oil-house. Additional storage space was provided by the contractor on 32d Street just west of First Avenue by renting three old buildings and the yards in the rear of them and of the Railroad Company's cement warehouse adjacent. Here electric conduits, pipe, castings, and other heavy and bulky supplies were stored.

During excavation the headings were supplied with forced ventilation through 12-in. and 14-in. No. 16, spiral-riveted, asphalted pressure pipes, canvas extensions being used beyond the ends of the pipes. A

No. 4 American Blower, located at the top of each shaft and driven by a 15-h.p. General Electric motor, supplied the air.

A concrete-mixing plant was placed in each shaft, the mixer being located high enough to discharge into cars at about the level of the springing line of the arch. Above the mixers were the measuring hoppers set in the floor of a platform which was large enough to carry half a day's supply of cement. At the South Shaft the cement was delivered to this floor from the loading platform through a spiral steel chute; at the North Shaft it was lowered in buckets by the telpher. The sand and stone were drawn into the hoppers through short chutes from the base of the storage bins which occupied the remaining height of the shaft—about 50 ft. At the South Shaft the bins were of concrete and steel, about 6 by 12 ft. in section, and attached to the central wall of the caisson. Sand and stone were delivered into them from dump-wagons on the loading platform. At the North Shaft steel-plate bins were used, and were supplied with material by the buckets handled by the telpher. The mixers were No. 5 Smith, belt-connected to 25-h.p. motors, and about 0.8 cu. yd. of concrete was mixed at a batch. The concrete cars were steel side-dumpers of the Wiener or Koppel type.

In order to be able to continue concreting during the winter, when neither sand nor stone could be obtained by water, practically all the space under the loading platforms in the South Shaft yards not occupied by the blacksmith shop was filled with these materials, which were placed in storage in the late fall.

Intermediate-Shaft Plant.—The air-compressing plant was located at the rear of the 33d Street Intermediate Shaft, and supplied air for driving the tunnels east and west from the Intermediate Shafts on both 32d and 33d Streets. Two compressors, the same as the large Laidlaw-Dunn-Gordon machine at First Avenue, were installed here, with a similar water-cooling tower.

Both shafts were on private property, owned by the Railroad Company, on the north side of the streets, and each was equipped with two telphers supported on timber trestles, similar to those at First Avenue. Here, however, the buckets were placed on wagons standing at the curb, as shown by Fig. 3, Plate LXXVIII.

Blowers for ventilation were installed at each shaft, as at First Avenue, and, after the excavation had proceeded some distance, small

FIG. 3.—HEADWORKS AT 33D STREET: INTERMEDIATE SHAFT.

FIG. 4.—LOADING SPOIL ON BARGES, 35TH STREET PIER.

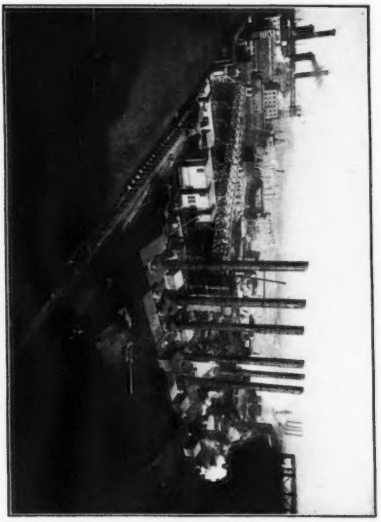

FIG. 1.—VIEW OF FIRST AVENUE PLANT.

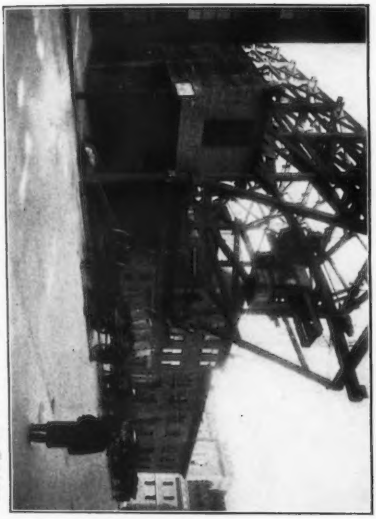

FIG. 3.—HEADWORKS AT 33D STREET: INTERMEDIATE SHAFT.

FIG. 2.—TELPHER STRUCTURE AND LOADING PLATFORM, FIRST AVENUE SHAFT.

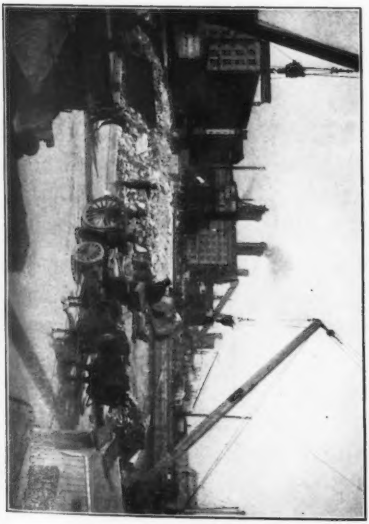

FIG. 4.—LOADING SPOIL ON BARGES, 35TH STREET PIER.

blacksmith shops, for sharpening drill steel and making minor repairs, were located in the tunnels near the shafts.

The concrete plant in each shaft was similar in arrangement to those at First Avenue, but the storage bins had wooden walls made of 2 by 4-in. and 2 by 6-in. scantling nailed flat on each other.

The contractor's office on 33d Street backed up against the 32d Street shaft site, and the basement was used as a storeroom for supplies for both shafts.

After the decision to do part of the work between Sixth and Seventh Avenues in open cut, an 8-in. air main was laid in 33d Street to the West Shafts, and air was supplied from the Intermediate Shaft for work on both streets in that neighborhood.

West-Shaft Plant.—West of Sixth Avenue, between 32d and 33d Streets and adjacent to the open-cut sections, the Railroad Company obtained from the Hudson and Manhattan Railroad Company the use of a large area from which the buildings had recently been removed, and gave the use of it to the contractor. This was of great value in prosecuting the west end of the work. The two West Shafts were located in the streets and were supplied with short timber trestles similar to those at the Intermediate Shafts. One telpher was taken from each of the Intermediate Shafts to operate at each of the West Shafts. In addition, a number of stiff-leg derricks were set up along the open-cut section, and were operated by Lidgerwood or Lambert air hoisting engines, or by electric motors, as circumstances dictated. A 15-ton Bay City locomotive crane was also used along part of the open-cut work on 32d Street.

Several concrete plants were installed at points along the open-cut section, and were moved from place to place, the same general arrangement being adopted as at the plants already described. No. 3 and No. 4 Ransome mixers were used, and were generally set up at about the level of the top of the arch. The sand and stone storage bins were made of scantlings spiked together, and were necessarily rather shallow on account of the proximity of the tunnels to the street surface.

Thirty-fifth Street Pier.—For the receipt and disposal of materials at the 35th Street pier, four stiff-leg derricks, operated by electric hoisting engines, were installed. Two were used in lifting the muck buckets from the wagons and dumping their contents on the scows for final disposal (Fig. 4, Plate LXXVIII); and the other two

were fitted with clam-shell buckets for unloading sand and broken stone from barges and depositing the materials in large hoppers, from which they were drawn into wagons for transportation to the various concrete plants. A large part of the cement (all of which was supplied by the Railroad Company) was also unloaded at the 35th Street pier and hauled directly to the work, the surplus being stored temporarily in the Company's cement warehouses on 32d, 33d and 35th Streets, near First Avenue, from which it was drawn as required. On the dock was located the main powder magazine, a small concrete structure. Considerable use was also made of neighboring piers for unloading electric conduits, lumber, steel, etc.

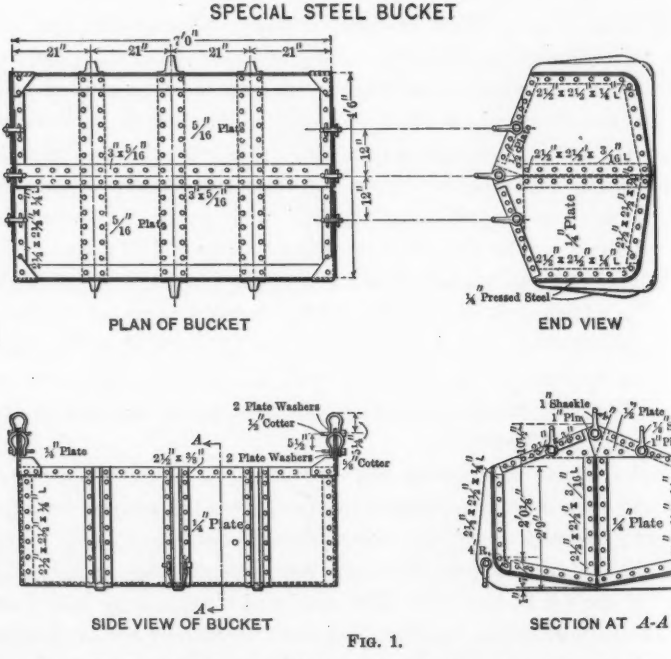

FIG. 1.

Tunnel Plant.—The spoil buckets, designed by D. L. Hough and George Perrine, Members, Am. Soc. C. E., were a novel feature of the work. These buckets are shown in detail in Fig. 1 and various photographs. They were of 3 cu. yd. capacity and were split longitudinally, the two halves being pinned at the apices of the ends. For lifting, they were suspended from eyes at that point, and, when dumping, trip

FIG. 3.—FIRST SECTION OF CONCRETE LINING AT FIFTH AVENUE.

FIG. 4.—TIMBERING AND RUBBLE MASONRY OVER I-BEAMS.

PLATE LXXIX.
PAPERS, AM. SOC. C. E.
OCTOBER, 1909.
BRACE AND MASON ON
CROSS-TOWN TUNNELS: PENNSYLVANIA RAILROAD.

FIG. 1.—AIR-OPERATED STEAM SHOVEL USED IN TUNNEL.

FIG. 2.—TIMBERING IN TOP HEADINGS ABOVE I-BEAMS.

FIG. 3.—FIRST SECTION OF CONCRETE LINING AT FIFTH AVENUE.

FIG. 4.—TIMBERING AND RUBBLE MASONRY OVER I-BEAMS.

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ropes were hooked into eyes at the bottom of each side; lifting the trip ropes or lowering the hoisting rope split the bucket, as shown in Fig. 4, Plate LXXVIII, and dumped the contents. They were transported in the tunnel on flat cars, and in the street on wagons, both cars and wagons being provided with cradles shaped to receive the bottom of the bucket.

In the tunnels the loading was done with air-operated steam shovels, four (Model 20) Marion shovels being used at various points of the work. In Fig. 1, Plate LXXIX, one of these is shown loading the bucket. The cars were hauled by General Electric, standard, 10-ton, mine locomotives, the current for which was taken at 220 volts from a pair of No. 00 copper trolley wires suspended from the roof of the tunnel. The collector was a small four-wheeled buggy riding on the wires and connected to the locomotive by several hundred feet of cable wound on a reel for use beyond the end of the trolley wire. Two 8½-ton, Davenport, steam locomotives were also used in 32d Street, toward the end of the work, after the headings had been holed through and the tunnels would quickly clear themselves of gas and smoke. The steam shovels were supplemented by two Browning, 15-ton, locomotive cranes, which handled the spoil in places where timbering interfered with the operation of the shovels. All tracks were of 3-ft. gauge throughout and laid with 40-lb. rails.

Practically all the heavy drilling was done with Ingersoll drills (Model E 52), the trimming being largely done with jap and baby drills. A large number of pumps were used at various points on the work, and practically all were of Cameron make, the largest ones at the shaft being 10 by 5 by 13-in. The grout machines were of the vertical-cylinder, air-stirring type.

Shaft Sinking.

The sinking of the Intermediate Shafts was the first work undertaken by the contractor.

The 33d Street Shaft was 34.5 ft. long, 21 ft. wide, and 83 ft. deep. The rock surface averaged 5 ft. below the ground surface. Sinking was started on July 10th, 1905, and was completed on October 3d, 1905, the rock being hard and dry. The average daily rate was 0.73 ft. and an average of 17.1 cu. yd. were excavated per day, with two shifts of 8 hr. each. The first shift started at 6 A. M. and the second at

2.30 P. M., ending at 11 P. M. These hours were adopted in order to avoid undue disturbance during the night.

Before blasting the first lift of rock, channel cuts 5 or 6 ft. deep were made along the sides of the shaft, in order to avoid damage to the walls of neighboring buildings. Timbering was required for a depth of only 10 ft. below the surface of the ground.

A drift, 30.6 ft. long, 17 ft. wide, and 27 ft. high, connected the south end of the shaft with the tunnels. The drift was excavated in three stages, a top heading and a bench in two lifts. While blasting the cut in the top heading, there was enough concussion to break glass in the neighboring buildings. The use of a radialax machine reduced the concussion somewhat, but it was very quickly abandoned on account of the length of time required for the drilling.

The construction of the 32d Street Shaft was quite similar to the one on 33d Street. It was 31.5 ft. long, 20.5 ft. wide, and 71 ft. deep. The depth of earth excavation averaged 19.5 ft. The rock in this shaft was seamy and not quite as hard or dry as that in 33d Street, and timbering was required for practically the full depth to the crown of the drift. Sinking was started on May 15th, 1905, and was completed on October 26th, 1905. The daily average rate was 0.30 ft. in earth and 0.52 ft. in rock. The drift was excavated in much the same manner as the one in 33d Street, but the rock being softer the radialax machine was not used.

TUNNEL EXCAVATION.

During the early part of the work, the contractor devoted his entire attention to the work of excavation. Nearly all the excavation east of Fifth Avenue was done before any of the lining was placed. At a number of points west of Fifth Avenue and at a few points to the east the nature of the rock was such that the two operations had to be done simultaneously.

Single-Tunnel Method.—For an average distance of 350 ft. west from the First Avenue Shafts there were four single tunnels. The rock was sound and comparatively dry. A top heading of the full size of the tunnel and about 8 ft. high was first driven. It was drilled by four drills mounted on two columns, and was blasted in the ordinary way. The bench was about 13 ft. high. Tripod drills, standing on the bench, drilled the usual holes, but, owing to the lack of head-room,

steels long enough to reach the bottom of the bench could not be used. Tripod drills were set as low as possible at the foot of the bench and drilled lifting holes. These holes were inclined downward from 10° to 15° to the horizontal, and were spaced to converge at the location of the drainage ditches. The heading was usually driven from 10 to 20 ft. in advance of the bench. At this distance a large part of the muck from the heading was shot backward over the bench. In the single tunnels the muck was loaded by hand.

Twin-Tunnel Methods.—From the end of the single-track tunnel westward to Fifth Avenue on 33d Street, and to Madison Avenue on 32d Street, with some exceptions, each pair of tunnels was excavated for the entire width at one operation. Three different methods of work were extensively used. They were the double-heading method, the center-heading method, and the full-sized-heading method, and these differed only in the manner of drilling and blasting. The bench was usually within 10 or 15 ft. of the face of the heading, and was drilled and fired in the same way as in the single tunnels. After the installation of the permanent plant, most of the muck was handled by steam shovels.

In the double-heading method, shown on Plate LXXVII, the top headings for each tunnel of the pair were driven separately, leaving a short rock core-wall between them. The headings were drilled from columns in the manner described for the single tunnels. The temporary rock dividing wall between the headings was drilled by a tripod drill on the bench of one of the headings, and was fired with the bench.

In the center-heading method, also shown on Plate LXXVII, only one heading was driven. It was rectangular in shape, about 8 ft. high and 14 ft. wide. It was located on the center line between the tunnels. In general, the face was from 6 to 12 ft., or the length of one or two rounds, in advance of the remainder of the face at the top. The center heading was drilled by four drills mounted on two columns. By turning these drills to the side, they were used for holes at right angles to the line of the tunnels, by which the remainder of the face of the heading was blasted. By turning the drills downward, the bench holes under the center heading were also drilled. The center heading explored the rock in advance of the full-width heading, and gave a good idea as to the care needed in firing.

For the full-width-heading method, Fig. 2, ten drills were mounted

on five columns set abreast across the face. Holes were drilled to form a cut near the center line between the tunnels. The remainder of the holes were located so that they would draw into the cut. The bench was frequently drilled from the same set-up of columns by turning the drills downward. In sound rock this method proved to be the most rapid of any.

Practically all trimming was left until immediately before the concreting. It was then taken up as a separate operation, but proved to be costly and tedious, and a hindrance to the placing of the lining.

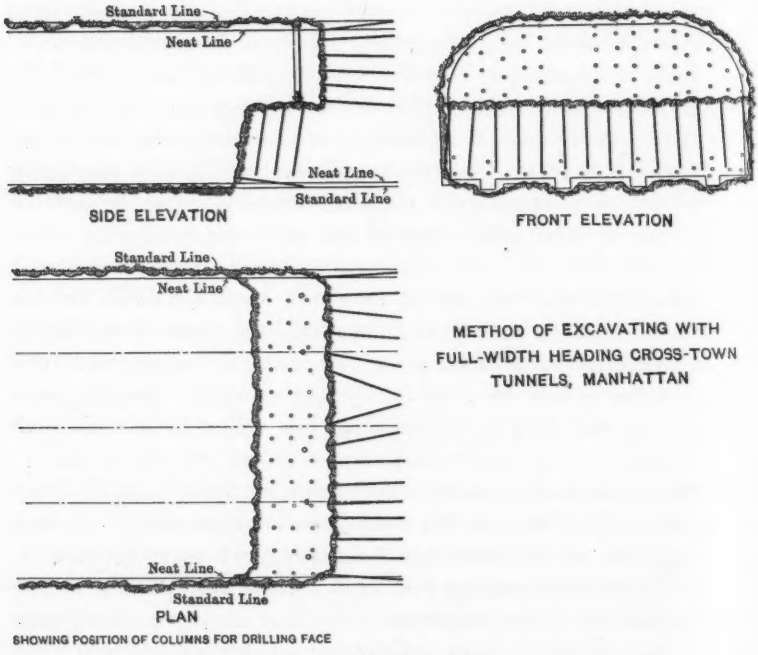


FIG. 2.

Materials Encountered.—All the rock encountered was the familiar Hudson schist, but it varied widely in its mineral constituents and in its physical characteristics. In many places where the rock surface was penetrated, a fine sand was found that was probably quicksand. The material above the rock in the open-cut sections was mostly sand.

The concurrence of the watercourse, shown on Gen. Viele's map of

Manhattan Island (Plate LXI*), with the points where difficulties in the construction of the tunnels were encountered has been noted in a previous paper.

In all cases where the course of this ancient stream was crossed (except at its final intersection of 33d Street), the rock was found to be very soft and disintegrated, a large quantity of water was encountered, and heavy timbering was required. The construction at these localities will be taken up later. In addition, disintegrated rock, but of a less troublesome character, was invariably met under the depressions in the rock surface developed by the borings from the streets and test holes from the tunnels. Many of these places required timbering, and no timbering was elsewhere necessary except at the portals. These coincident conditions were especially marked in 32d Street, which for a long distance closely adjoins the course of the former creek.

Disposal of Spoil.—The materials excavated from the tunnels were dumped at the 35th Street pier on barges furnished by the Railroad Company under another contract, and were towed to points near the Bayonne peninsula where the spoil was used principally in the construction of the Greenville Freight Yards and the line across the Hackensack Meadows to the tunnels. Details of this work will be given in a subsequent paper. After December, 1907, when the excavation was about 85% completed, the contractor furnished the barges and effected the complete disposal of the spoil.

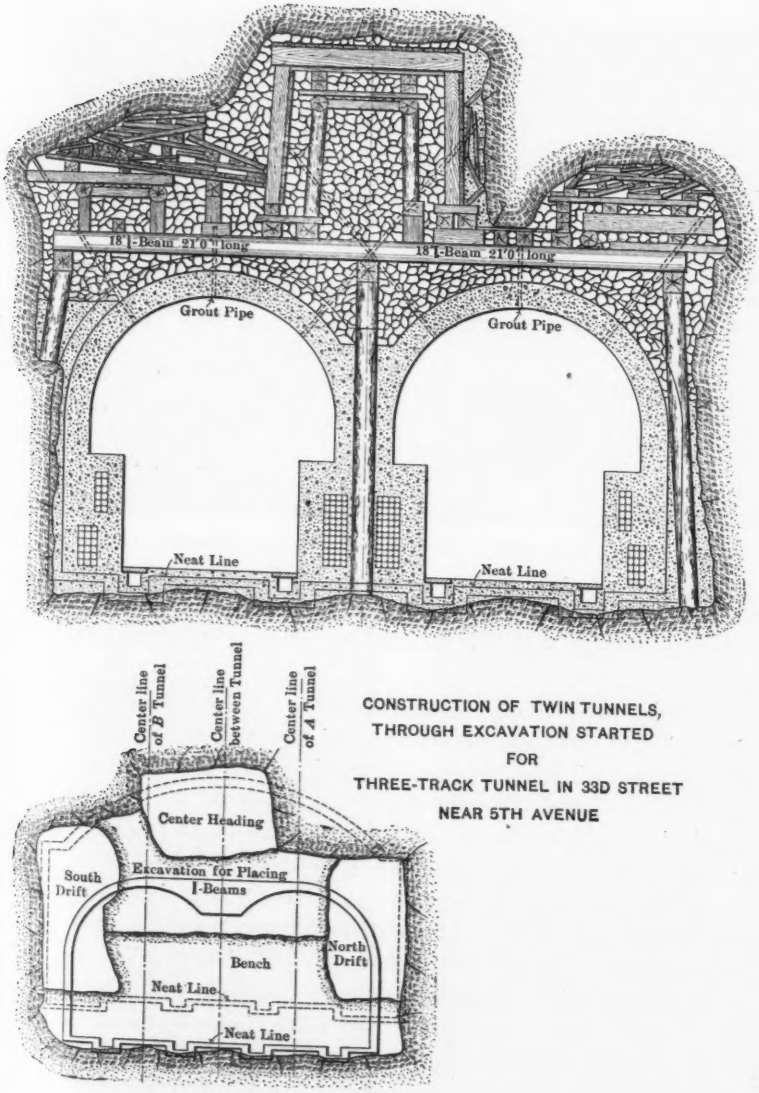
Difficulties of Excavation.—As stated in a previous paper, the excavation of the Twin Tunnel in 33d Street was continued westward to the west line of Fifth Avenue on the original grade. At that point the contractor started three drifts in the three-track section. The relation of the drifts to each other and to the cross-section are shown by Fig. 3. The center heading was driven a little in advance of those on the sides. At a distance of 65 ft. west of Fifth Avenue the rock surface was broken through in the top of the heading, and a very fine sand was encountered. For some distance east of this point the rock was badly disintegrated, and the heading required timbering. Through the soft material, tight lagging was placed on the sides and roof of the heading, and the face was protected by breast boards. There was a moderate flow of water through the cracks, and, in spite of every effort, some of the fine sand was constantly carried into the heading.

* *Proceedings*, Am. Soc. C. E., for September, 1909.

In one or two instances considerable ground was lost at the face. On the evening of December 14th, 1906, as a heavy coal wagon was passing along 33d Street above the heading, the rear wheels dropped through the asphalt pavement. An examination disclosed a cavity under the pavement about 14 ft. long, 12 ft. wide and 14 ft. deep. Evidently, the fine sand had gradually settled into the voids caused by the loss of material at the face, and the settlement broke the brick sewer over the heading. The sewer was temporarily repaired, and the hole in the street was filled before morning. A tight bulkhead was built acrosss the heading, and work was abandoned at that point. The north drift was advanced to a point 108 ft. west of Fifth Avenue where sand was also encountered and a considerable run occurred. After that time all work on the three-track section was discontinued.

The Company then took up the consideration of changes in plan. To determine the difficulties of driving a Twin Tunnel at a lower elevation, an exploration drift, 8 ft. high and 12 ft. wide, was driven on the center line of the street as a top heading on the proposed new grade. Test holes were drilled above this heading and to the sides. The results indicated that there was sufficient rock cover of fair quality to enable the Twin Tunnel to be driven without great risk. The new plan (continuing the Twin Tunnel westward at a lower grade) was adopted in March, 1907, and work was immediately resumed at Fifth Avenue.

The relation between the cross-sections under the old and new plans at that point is shown by Fig. 3. Before the new section was excavated it was necessary to support the timber work in the old headings. The plan adopted is also shown by Fig. 3. The rock was excavated under the center heading, as shown in cross-section, for a length of about 3 ft. A girder composed of two 18-in. I-beams was then put in position over each line and supported on the sides by posts. The ends at the center lines between the tunnels were supported on short posts bearing on the rock bench. The support of the timbering in the headings was then transferred to the girders by additional posts. Blocking was also inserted between the tops of the beams and the rock walls between the headings. Fig. 2, Plate LXXIX, gives a good idea of the timber work in the top headings above the I-beams. When the roof had been made secure, the removal of the bench was begun. As the work advanced it was necessary to replace the short posts at



CONSTRUCTION OF TWIN TUNNELS, THROUGH EXCAVATION STARTED FOR THREE-TRACK TUNNEL IN 33D STREET NEAR 5TH AVENUE

FIG. 3.

the center of the tunnel by others of full height, and there was considerable settlement in the I-beams during this operation. When the bench had been removed to a point 61 ft. west of Fifth Avenue, settlement was detected in the street surface above. Bench excavation was suspended and a section of the permanent lining, 35 ft. long, was placed. The space between the lining and the beams and between the beams and the roof was filled with rubble masonry. Grout pipes were built into the masonry and later all voids were filled with grout. Fig. 3, Plate LXXIX, shows the first section of the concrete lining completed and part of the rubble in place; and Fig. 4, Plate LXXIX, shows details of the work above the tunnels. A second section of bench was next removed and more lining was placed. Work was continued in this way until all the roof at the old three-track headings had been secured. In this portion of the work the posts were embedded in the concrete.

Between Fifth and Sixth Avenues there were two more sections of bad rock where it was necessary to support the roof with steel beams. At these latter points there were no complications with the excavation for the Three-Track Tunnel, and the work was much simpler. To avoid leaving the center posts in the permanent work, two rows of temporary posts were placed, as shown by Fig. 1, Plate LXXX, the center wall and skewback were built, and the posts were removed, as shown by Fig. 2, Plate LXXX, before placing the remainder of the lining.

In 32d Street the normal progress of the excavation was frequently interrupted by encountering soft and unsound rock. In the excavation between the East River and the Intermediate Shafts it was possible to overcome these conditions by temporarily narrowing the excavation on one side and supporting the roof on 16 by 16-in. transverse timbers caught in niches in the rock at the sides, leaving sufficient room for the steam shovel to work through. In order to save time, the height of the excavation was not increased before placing these timbers, so that, previous to the concreting, they all required to be raised to clear the masonry lining and were then supported on posts on the center line between the tunnels. This permitted the remainder of the excavation to be made, and such additional timbering as was required was placed. At most of these sections a brick arch and water-proofing were used, on account of the presence of water. In certain places the center line posts were buried in the core-wall, and, in order to permit the placing

FIG. 1.—DOUBLE ROW OF POSTS UNDER I-BEAMS, SUPPORTING ROOF IN BAD ROCK SECTION.

FIG. 3.—TIMBERING IN FULL-WIDTH HEADING OF THREE-TRACK TUNNEL.

FIG. 2.—CENTER WALL AND SKEWBACK UNDER I-BEAMS, AFTER REMOVAL OF DOUBLE ROW OF POSTS.

FIG. 4.—UNDERPINNING WALLS IN OPEN-CUT SECTION.

of the water-proofing, were then cut off one by one flush with its top as the load was transferred to the completed masonry. In other cases the load was transferred to posts clear of the masonry and the center line posts were entirely removed. Under such conditions the normal concrete methods, to be described later, could not be used, and special forms were substituted.

In this section of the work the most serious difficulties were encountered near Fourth Avenue a short distance east of the Intermediate Shaft, and beneath the site of the old pond shown on Gen. Viele's map. The rock cover was known from the boring to be very thin, and the presence of the subway overhead caused some anxiety. The excavation was at first taken out to practically full width and timbered, but the rock became so treacherous that the heading was narrowed to a width sufficient for one tunnel only. With this span the rock in the roof held without timbering. As the masonry lining approached, sufficient trimming was done to permit the placing of the core-wall and one arch. Above the completed core-wall and brick arch the voids were filled solid with rubble masonry to give an unyielding support to the roof. The excavation of the remaining width of tunnel was then undertaken. Near the west side of Fourth Avenue, the excavation broke out of rock at the top, and fine sand and gravel with a large quantity of water were encountered. The work of excavation was arduous, and proceeded very slowly, on account of the care with which it was executed. Only a small amount of sand entered the tunnel, but the lining was placed as soon as the excavation was completed. Rubble masonry packing and grout ejected through pipes built into the arch were used to fill the voids above the roof. As a further precaution against the settlement of the subway, 2-in. pipes were washed down from the street above the point where soft ground was exposed in the roof of the tunnel, and through them grout was forced into the ground at various depths. Careful levels show that no settlement of the subway has taken place.

West of the Intermediate Shaft the tunnel was excavated for full width until bad rock was encountered about 60 ft. west of Madison Avenue. (See Gen. Viele's map, Plate LXI.*) Timbering was used for a short distance, and then the heading and bench were narrowed to 18 ft., and steam-shovel excavation was abandoned. As the heading

* *Proceedings*, Am. Soc. C. E., for September, 1909.

advanced the rock grew steadily softer, the difficult conditions in this locality culminating when a slushy disintegrated feldspar was met, requiring poling and breasting. Thereafter the rock improved markedly, but near the east side of Fifth Avenue its thickness above the roof was found to be only 1½ ft., and the advance was stopped, pending a decision as to a change of plan.

After some delay, an exploration drift, similar to the one already described, was driven through to Sixth Avenue, and a change in plan was made, substantially the same as for the 33d Street tunnels. Enlargement to full size was at once started, but, for 400 ft. the rock was very soft and poor, and required extremely careful handling. The exploration drift was widened out to the full Twin-Tunnel width, and I-beams were placed and supported, in much the same manner as in 33d Street. The rock was so soft that it was frequently necessary to drive poling boards ahead as the face was mined out with picks and shovels. The load was very heavy, and the work the most difficult encountered in the tunnels.

After this stage of the enlargement was reached, the excavation of the bench and the placing of the lining proceeded alternately, with the I-beams temporarily supported on long posts while the concrete core-wall was being built. Considerable settlement took place while shifting the posts, and eventually showed on the street surface and in the adjacent sidewalk vaults, but no damage was done to the structural portions of the buildings.

While the above work had been going on westward from Fifth Avenue, the excavation of the Twin Tunnel eastward from the end of the open-cut section at Sixth Avenue had been proceeding rapidly, and, toward the end of the difficult Fifth Avenue work, it was being attacked from both directions.

Progress of Excavation.

Owing to the numerous sections of poor rock, interspersed throughout the work with stretches of sound rock, the progress of the excavation was very irregular, especially in 32d Street. The rate of excavation in good ground is shown in Table 1. In the sections of bad ground, the operations of excavation, timbering, and lining were often carried on alternately, and it is impracticable to include them in the table.

Three-Track Tunnel Excavation.

When it became evident that the work through the Fifth Avenue section would be extremely slow, shafts were sunk in each street between Sixth and Seventh Avenues. The shafts, as shown on Plate LXVI,* were located in the streets, but in such a way as to block only half of the roadway. At the same time it was decided to construct in open cut about 200 ft. of the Three-Track Tunnel at the west end of the contract in 32d Street, where the rock surface was below the top of the tunnel. It was hoped that the remainder of the work could be built without opening the street, but further investigation showed that this was impracticable, and eventually all the Three-Track Tunnel in 32d Street, except 120 ft. east of the shaft, was built in open cut.

Thirty-second Street Work in Tunnel.—Following the sinking of the shaft, a drift was driven across the street at the crown of the tunnel, and a top heading on the south side was excavated in both directions. Frequent cross-drifts to the north side showed that the rock was nowhere very sound and that, except for a short distance east of the shaft, it was distinctly unfavorable for the wide Three-Track excavation. In this stretch the north ends of these cross-cuts were connected by a second heading, and wall-plates and sets of three-segment arch timbering were set up to support the roof of the drifts. The cross-cuttings were gradually widened and timbered until the entire excavation had been made down to the level of the wall-plates, as shown in Fig. 3, Plate LXXX. The bench was then excavated in two lifts, leaving the wall-plates supported on narrow longitudinal berms, which were removed in short sections to permit the placing of posts under the wall-plates.

Thirty-second Street Open-Cut Work.—Before actual open-cut excavation was started, all buildings facing it were underpinned to rock. For this purpose, a trench was dug along the face of the buildings and of the same depth as their cellars. Holes were cut in the front foundation walls through which long needle-beams (Fig. 4, Plate LXXX) were inserted and jacked up on blocking placed on the cellar floor and in the trench, until the weight of the building had been taken off its foundations. A close-sheeted trench was then sunk to rock under the front building walls, and a light rubble masonry

* *Proceedings*, Am. Soc. C. E., for September, 1909.

TABLE 1.—PROGRESS AND METHODS

THIRTY-THIRD STREET.

Type of excavation.	Tunnels.	Worked from:	Dates.	
			From	To
Full-sized single tunnel.....	B	1st Ave. shaft.	Feb. 28, 1906.	May 12, 1906.
Full-sized single tunnel.....	A	1st Ave. shaft.	Feb. 28, 1906.	Apr. 30, 1906.
Full-sized twin tunnel	A and B	1st Ave. shaft.	Aug. 23, 1906.	Jan. 5, 1907.
Full-sized twin tunnel	A and B	Intermediate shaft. (West of shaft.)	Apr. 4, 1906.	Oct. 31, 1906.
Full-sized twin tunnel	A and B	Intermediate shaft. (East of shaft.)	Apr. 4, 1906.	Oct. 31, 1906.
Full-sized twin tunnel	A and B	Intermediate shaft. (East of shaft.)	Nov. 1, 1906.	Dec. 26, 1906.
Exploration drift	A and B	Intermediate shaft. (West of shaft.)	Mar. 1, 1907.	July 23, 1907.
Twin tunnel. Enlargement of exploration drift.....	A and B	West shaft. (East of shaft.)	Sept. 6, 1907.	Dec. 4, 1907.

THIRTY-SECOND STREET.

Full-sized single tunnel.....	C	1st Ave. shaft.	Jan. 25, 1906.	Apr. 30, 1906.
Full-sized single tunnel.....	D	1st Ave. shaft.	Jan. 27, 1906.	Apr. 30, 1906.
Full-sized twin tunnel	C and D	1st Ave. shaft.	May 22, 1906. Aug. 11, 1906.	July 24, 1906.* Nov. 29, 1906.
Narrowed twin tunnel	C	Intermediate shaft. (East of shaft.)	Mar. 19, 1906.	May 28, 1906.
Narrowed twin tunnel	C	Intermediate shaft. (East of shaft.)	May 29, 1906. July 18, 1906. Aug. 12, 1906. Jan. 15, 1907. Feb. 17, 1907.	July 3, 1906.* July 31, 1906. Nov. 23, 1906. Feb. 5, 1907. Mar. 21, 1907.
Full-sized twin tunnel	C and D	Intermediate shaft. (West of shaft.)	Dec. 1, 1905.	May 10, 1906.
Exploration drift	C and D	Intermediate shaft. (West of shaft.)	Feb. 1, 1907.	Sept. 13, 1907.
Twin tunnel. Enlargement of exploration drift...	C and D	Eastward from open cut.	Feb. 1, 1908.	Feb. 14, 1908.
Twin tunnel. Enlargement of exploration drift...	C and D	Eastward from open cut.	Feb. 15, 1908.	Apr. 14, 1908.

*Time and distance omitted while working through timbered stretches.

OF EXCAVATION IN GOOD GROUND.

Time elapsed, in days.	Length of tunnel excavated, in linear feet.	Average advance per day, in linear feet.	Methods and conditions.
74	346	4.7	Top heading and bench. Muck loaded by hand.
62	255	4.1	" " " " " " " "
136	789	5.8	Top full-width heading and bench. Muck loaded by steam shovel. Working exclusively on this heading.
210	730	3.5	Top center heading and bench. Muck loaded by steam shovel. Working alternately in headings east and west of the shaft.
210	783	3.7	Top center heading and bench. Muck loaded by steam shovel. Working alternately in headings east and west of the shaft.
56	311	5.5	Top full-width heading and bench. Muck loaded by steam shovel working exclusively on this heading.
145	947	6.5	Exploration drift about 9 ft. by 12 ft. Mucking by hand. Fourteen timber bents were placed in March, and seven in April, 1907.
89	603	6.8	Drift excavated to full width and bench. Muck loaded by steam shovel.
95	367	3.9	Top heading and bench. Muck loaded by hand.
93	354	3.8	" " " " " " " "
173	810	4.7	Double heading and bench. Muck loaded by steam shovel. Stretches aggregating 200 ft. narrowed to about 25 ft. and later enlarged are included.
70	58	0.8	Excavation about 30 ft. wide. Top full-width heading and bench. Muck loaded by hand. Steam shovel not installed.
208	1 206	5.8	Excavation about 30 to 35 ft. wide. Top full-width heading and bench. Muck loaded by steam shovel.
161	225	1.4	Double heading and bench. Part of the muck handled by hand and part by steam shovel.
225	1 033	4.6	Exploration drift about 10 ft. by 13 in. Muck loaded by hand. 14 ft. timbered.
14	65	4.6	At portal of twin tunnels. Drift excavated to full width and bench. Muck loaded by hand. 12 ft. timbered.
59	524	8.9	Drift excavated to full width and bench. Muck loaded by steam shovel. Full-width tunnel timbered for 26 ft. independently of the main excavation.

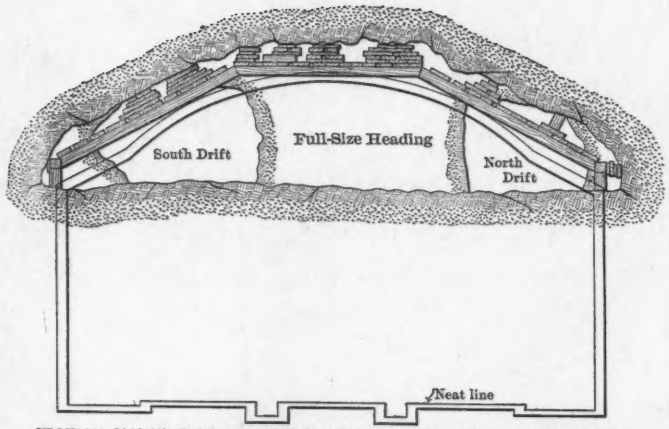
retaining wall was built in it to support the building permanently. Frequently, the excavation for the underpinning wall, which was taken out in sections from 30 to 40 ft. long, and in places was carried to a depth of 40 ft., was very troublesome on account of the large quantity of water encountered and the fineness of the sand, which exhibited a tendency to flow when saturated.

The Elevated Railroad columns in Sixth Avenue, near the north and south lines of 32d Street, were underpinned in a manner similar to the building foundations, while those on the center line of the street were supported by girders riveted to them close under the track level. The girders in turn were supported on posts footed on the new underpinning of the adjacent columns. On the completion of the tunnels, concrete piers were built up from the roof of the tunnel to form a permanent foundation for the center-line columns. The area to be excavated under Sixth Avenue was enclosed by a rubble masonry retaining wall constructed in a trench.

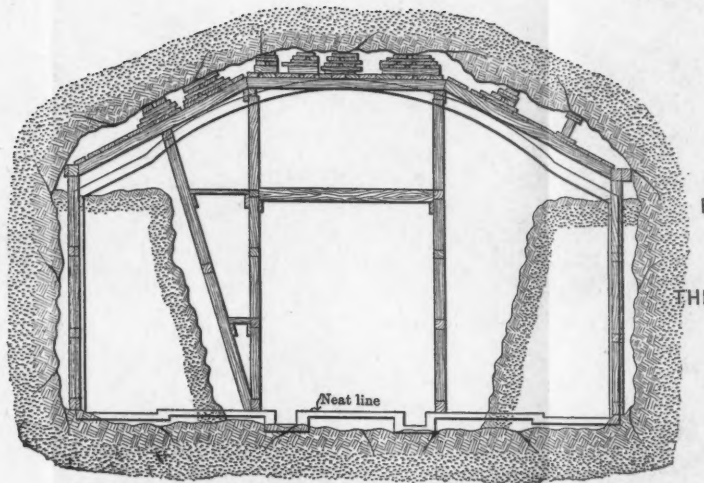
Open-cut excavation was started by planking over the street on stringers resting on transverse 12 by 12-in. caps. The caps were gradually undermined and supported on temporary posts which were then replaced by short posts resting on 12 by 12-in. sills about 7 ft. below the cap. The operation was then repeated and the sill was supported on another set of short posts resting on a second sill. When the excavation had been carried down in this manner to the level of the top of the tunnel, diagonal 3 by 10-in. timbers were cut in between the posts and sills to form a species of double **A**-frame, the legs of which rested in niches cut in the rock and on posts carried up the face of the underpinning wall, and the whole was stiffened with vertical tie-rods. This construction is shown by Fig. 3, Plate LXXXII. The brick sewer was replaced temporarily by one of riveted steel pipe. This pipe and the water and gas pipes and electric conduits were suspended from the timbers as the pipes were uncovered.

Excavation in rock was made by sinking a pit to sub-grade for the full width of the tunnel and advancing the face of the pit in several lifts, the muck being blown over the slope and loaded into buckets at its foot.

The work was attacked at several places simultaneously, and the spoil was hoisted by derricks located at convenient points along the side of the cut.



SECTION SHOWING NORTH AND SOUTH HEADING, ALSO ENLARGEMENT TO SINGLE HEADING AND METHOD OF TIMBERING

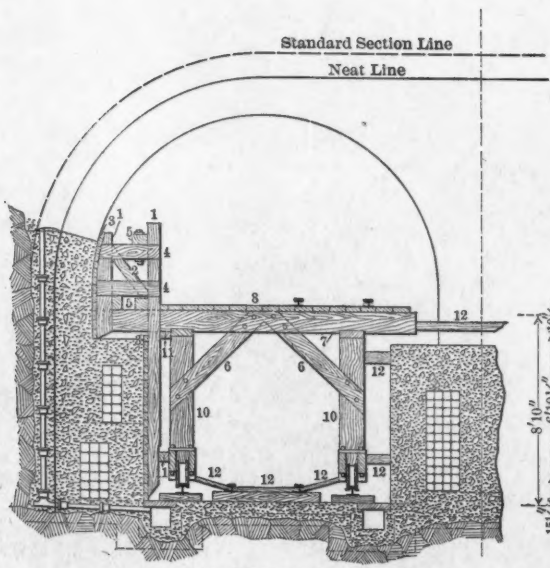


METHOD OF EXCAVATING BENCH, SHOWING CENTER EXCAVATION WITH TIMBERING ALSO ENLARGEMENT TO FULL SIZE AND TIMBERING

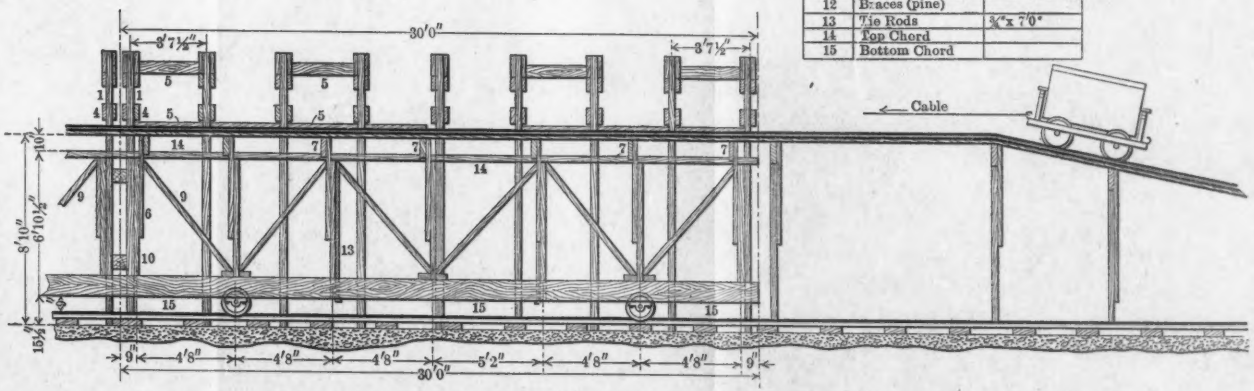
EXCAVATION AND TIMBERING IN HEAVY GROUND OF THREE-TRACK TUNNEL OF 33D ST.

CARRIAGE FORM FOR SIDE WALLS OF CROSS-TOWN TWIN TUNNELS

Piece	Description: Location	Size
1	Studs	4"x 8"
2	Diagonals	4"x 6"x 3'
3	Lagging	2"dressed 15'lg
4	Braces (oak)	2"x 8"x 3'
5	Jacking Timbers	6"x 6"
6	Knee Braces	2"x 10"x 6'6"
7	Floor Joists	3"x 12"x 14'6"
8	Flooring	2"x 10"
9	Diagonal Braces	3"x 6"
10	Posts	3"x 12"
11	Wedges	
12	Braces (pine)	
13	Tie Rods	¾"x 7'0"
14	Top Chord	
15	Bottom Chord	



CROSS-SECTION



LONGITUDINAL SECTION ON C. L. OF CARRIAGE

Thirty-third Street Work in Tunnel and Open Cut.—The West 33d Street Shaft was similar to the one in 32d Street, and was sunk during February, March, and April, 1907, through 10 ft. of earth, 21 ft. of soft rock, and 29 ft. of fairly hard rock. It was necessary to timber heavily the upper 30 ft. of the shaft. The timber later showed evidences of severe strain, and had to be reinforced.

As soon as the shaft excavation was deep enough, a drift was driven part way across the tunnels, and top headings were started both east and west to explore the rock. The heading to the west was divided into two drifts, as shown on Plate LXXXI. These two drifts were continued to the west end of the contract, and were then enlarged to a full-sized heading and timbered, as shown on Plate LXXXI and Fig. 3, Plate LXXX. The rock near the shaft contained many wet rusty seams, and settlement was detected in the segmental tunnel timbering soon after the widening of the heading was completed. Short props were placed under the timbers, and the street surface was opened with a view of stripping the earth down to the rock and thus lightening the load on the timbering. Street traffic was maintained on a timber structure with posts eventually carried down to the rock surface, and the walls of the buildings on the north side of the street were underpinned to rock. The settlement of the tunnel timbering was checked for a time, and the bench was excavated as shown on Plate LXXXI. In this work the cut in the center was first made, and the short props were replaced by struts, as shown; after this the berms were removed and the side posts were placed. While building the brick arches, holes were left in the masonry around the struts. After the masonry had hardened, piers were built on the arches to support the segmental timbers. The struts were then removed and the openings filled with masonry. The voids above the arch were packed with rock and afterward thoroughly grouted.

The timbers near the shaft continued to settle, and, although they had been placed from 9 to 12 in. above the level of the top of the masonry, by October 1st, they encroached 9 in. within the line of masonry. It was then decided to remove the rock for a distance of 48 ft. west of the shaft, and build this portion of the tunnel in open cut. The posts supporting the deck forming the street surface were replaced by an **A**-frame structure similar to that developed for the 32d Street open cut, without interruption of the street traffic.

After making the open cut to the westward of the shaft, there was a slip in the rock north of and adjoining the shaft. Fortunately, the timbers did not give way entirely, and no damage was done. The open cut was extended eastward for a distance of 46 ft., making the total length of tunnel built in open cut on this street 94 ft.

East of the shaft, for a distance of about 125 ft., the rock was broken and could not be excavated to full size without timbering the roof, but between this section of poor rock and those already mentioned in connection with the work at Fifth Avenue, there was a stretch of 600 ft. of good rock where all the spoil was handled with a steam shovel.

Twin-Tunnel Lining.

The masonry lining for the tunnels was not started until the late fall of 1906, after excavation had been in progress for a year and a half. At that time concreting was started in the single tunnels westward from the First Avenue Shafts, and by spring was in full swing in the Twin Tunnels.

The plans contemplated the use of a complete concrete lining except where large quantities of water were encountered; in which case the arches, beginning at a point 15° above the springing line, were to be built of vitrified paving brick. By reference to Plate LXIV* it will be seen that the water-proofing, which in the concrete-roof tunnels extended the full height of the sides to the 15° line, was carried in the brick-roof tunnels completely around the extrados of the arch. The cross-sections also show the location of the electric conduits which were buried in the mass of the side and core-walls and which limited the height to which the concrete could be carried in one operation.

The same general scheme of operations was used wherever possible throughout the Twin-Tunnel work, but was subject to minor modifications as circumstances dictated. Concrete was first deposited in the bottom, to the grade of the flow line of the drains; after it had set, collapsible box forms, of 2-in. plank with 3-in. plank tops, were laid on it to form the ditch and the shoulders for the flagstone covers. The track, which had previously been blocked up on the rock between the ditches, was raised and supported on the ditch boxes above the finished floor level. At the same time, light forms were braced from the ditch

* *Proceedings*, Am. Soc. C. E., for September, 1909.

boxes to the grade of the base of the low-tension and telephone-duct bank. After depositing the concrete to this level, the telephone ducts were laid.

The forms for the water-proofing or sand-wall up to the 15° line and for the main side-walls and core-walls were built in 30-ft. panels and were supported on carriages, which, traveling on a broad-gauge track above the ditches, moved along the tunnel, section by section, as the work advanced. The panels were hung loosely from joists carrying a platform on the top chord of the carriage trusses, and were adjusted transversely by bracing and wedging them out from the carriage. The small forms for the refuge niches, ladders, etc., were collapsible, and were spiked to the main panel forms just previous to the deposition of the concrete. The concrete was deposited from the platform on top of the carriage, to which the cars were elevated in various ways. Plate LXXXI shows the details of the carriages, and is self-explanatory.

The concrete for the sand-walls and the core-wall, to the level of the sidewalk, was deposited at the same time; two carriages in each tunnel, placed opposite each other, forming a 60-ft. length, were used at each setting. The floor section of the 4-in. tile drains had been laid with the floor concrete, and, as the sand-wall concrete was deposited, the drains were brought up simultaneously, broken stone being deposited between the tile and the rock to form a blind drain and afford access to the open joints of the tile for the water entering the tunnel through seams in the rock. The drains were spaced at intervals not exceeding 25 ft., depending on the wetness of the rock, and were placed at similar intervals in the core-wall under the lowest projecting points of the rock on the center line between the tunnels. A small ditch lined with loose 6-in. vitrified half pipe was provided in the top of the sand-wall to collect the water from the extrados of the arch and lead it to the top of the drains. Great difficulty was experienced in maintaining these drains clear, and, on completion of the work, a large amount of labor was expended in removing obstructions from the floor sections, the only portion then accessible.

After water-proofing the sand-walls and laying the low-tension ducts, a second pair of carriages, with panels on one side only, for 60 ft. of side-wall and skewback to the 15° line, were set and braced against the core-wall. These forms are shown in connection with the

carriage on Plate LXXXI. They were concreted to the base of the high-tension duct bank, and, after the concrete had hardened and the bank of ducts had been laid, the concreting was completed in a second operation.

In places where the roof was supported temporarily by posts and heavy timbering, such as at Fifth Avenue, the form carriages could not be used, and special methods were devised to suit the local conditions. Usually, the panels were stripped from the carriages and moved from section to section by hand, and, when in position, were braced to the timbering.

The arch centers were built up of two 5 by 3 by $\frac{3}{8}$-in. steel angles, and, when set, were blocked up on the sidewalks opposite each other in the two tunnels. A temporary platform was laid on the bottom chord angles of the ribs, on which the concrete was dumped, the same as on the form carriages. The lagging used was 3 by 3-in. dressed pine or spruce 16 ft. long, and was placed as the concreting of the arch proceeded above the 15° line on the side-wall and above the sidewalk on the core-wall. After the arch had reached such a height that the concrete could not be passed over the lagging directly from the main platform, it was cast on a small platform on the upper horizontal bracing of the centers, shown in Fig. 3, Plate LXXIX, and was thence shoveled into the work. In the upper part of the arch the face of the concrete was kept on a radial plane, and, when only 3 ft. remained to be placed, it was keyed in from one end, the key lagging being set in about 5-ft. lengths. The arches were concreted usually in 60-ft. lengths.

Where brick arches were used, the core-wall skewback was concreted behind special forms set up on the sidewalks, or the arch ribs and lagging were used for forms, and the brick arch was not started until after the concrete had set. In laying the brick in the arch, the five courses of the ring were carried up as high as the void between the extrados and the rock would permit and still leave a working space in which to place the water-proofing. This was usually not more than 3 ft., except on the core-wall side. The felt and pitch water-proofing was then laid for that height, joined to the previous water-proofing on the side-walls, and was followed by the brick armor course over the water-proofing and by the rock packing, after which another lift of brick was laid and the operations were repeated. The large

void (Fig. 1, Plate LXXXII) above the core-wall gave convenient access for working on top of the adjacent sides of the roof, and the keying of the arches and the water-proofing and rock packing above the core-wall were usually carried on from that point, the work progressing from one end.

The concrete for all work above the floor was dumped on the platform of the carriages, to which it was transported in the early part of the work in cars running on a high-level track laid on long ties, resting on the finished sidewalks. This arrangement, although requiring a large amount of timber for the track, permitted the muck to be carried out on the low-level track without interference. Later, when the advance of the heading had ceased and the heavy mucking was over, all concrete was transported on the floor level, and the cars were lifted to the carriage platforms by elevators and were hauled by hoisting engines up a movable incline. The latter method is shown by Fig. 3, Plate LXXIX.

Water-Proofing.—The water-proofing referred to above was in all cases felt and pitch laid with six thicknesses of felt and seven of pitch. The sub-contractor for the work was the Sicilian Asphalt Paving Company. All joints were lapped at least 1 ft., and, where work was suspended for a time and a bevel lap could not be made, the edges of the felt were left unpitched for 1 ft. and the newer work was interlaced with the old. This method was not always successful, however, on account of the softening of the unpitched felt on long-continued exposure to the water. The felt used was mainly "Tunaloid," together with some "Hydrex." It weighed about 12 lb. per 100 sq. ft. when saturated and coated on one side only, and contained about 25% of wool. The coal-tar pitch used had a melting point of 100° Fahr.

After the completion of the tunnel, the concrete arch showed some leakage and in places unsightly lime deposits. It was determined to attempt to stop these leaks by the application of a water-proof cement coating on the intrados of the arch. Extended experimental application of two varieties of materials used for this purpose—"Hydrolithic" cement and the U. S. Water-proofing Company's compound—have been made with apparent success up to the present time, and the results after the lapse of a considerable period are awaited with interest.

Duct Laying.—The position of the electric conduits, buried in the heart of the concrete walls, interfered greatly with the economical and

speedy placing of the lining, and their laying proved to be one of the most troublesome features of the work. The power conduits were single-way, with the bank for high-tension cables separated in the side-walls from the low-tension bank, as shown on Plate LXIV*. The conduits for telephone and telegraph service were four-way, and were located in the core-wall. All ducts had ¾-in. walls and a minimum clear opening of 3⅜ in. square, with corners rounded. They were laid with joints broken in all directions, and in about ½-in. beds of 1:2½ mortar. Flat steel bond-irons, 2 by ⅛ in., with split and bent ends, were placed in the joints at intervals of 3 ft. and projected into the concrete 3 in. on each side, tying together the concrete on opposite sides of the ducts. The joints were wrapped with a 6-in. strip of 10-oz. duck saturated with neat-cement grout, and, in addition, the power conduits were completely covered with a ½-in. coat of mortar to prevent the intrusion of cement and sand from the fluid concrete. The four-way conduits were plastered only over the wraps. Splicing chambers were provided at intervals of 400 ft.

Three-Track Tunnel Lining.

In the Three-Track Tunnels, a heavy brick arch was used for those portions constructed in tunnel, while, in the open-cut sections, the roof was of concrete. Both were completely water-proofed on the roof and sides, and in the tunnel sections the space above the brick roof was filled with rock packing. On account of the unstable nature of the rock encountered throughout, the voids in the packing were afterward filled with grout.

By reference to the cross-sections, Plate LXIV*, it will be seen that the haunches of the arch were tied together by steel I-beams anchored in the concrete, with the object of making the structure self-supporting in the event of the removal of the adjacent rock for deep cellar excavations. This construction materially influenced the contractor's method of placing the masonry lining.

After depositing the floor concrete, by the same method that was used in the Twin Tunnels, a timber trestle (Fig. 2, Plate LXXXII) was erected to the height of the underside of the I-beam ties, the posts being footed in holes, about 3 in. deep, left in the concrete floor

* *Proceedings*, Am. Soc. C. E., for September, 1909.

FIG. 1.—WATER-PROOFING OVER BRICK ARCHES.

FIG. 3.—METHOD OF STREET SUPPORT OVER OPEN-CUT EXCAVATION.

FIG. 2.—TRESTLE USED IN CONCRETING IN THREE-TRACK TUNNEL.

FIG. 4.—JUNCTION OF TWIN AND THREE-TRACK TUNNELS.

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to prevent slipping. In the open-cut sections the sand-wall forms were of undressed plank tacked to the studding and braced from the trestle; in the tunnel section they were spiked to the face of the posts supporting the timbering.

The side-wall forms were made up in panels about 3 by 10 ft., and were clamped to studs by U-shaped irons passing around the stud and bolted to the cleats on the back of the panels, the studs being braced from the trestle. The side-wall concrete was deposited in three sections. The first was brought up just above the sidewalk and formed the bench for the high-tension ducts; the second carried the wall up to the springing line. Before placing the third section the I-beam ties were set in position (Fig. 3, Plate LXXXII) on top of the trestle, and the reinforcing rods in the haunch of the arch were hung from them. The concrete was carried up to a skewback for the arch, as shown in the brick-roof cross-section (Plate LXIV*) and embedded the ends of the ties.

The centers for the arches stood on the I-beam ties, and the tops of the hangers, for the permanent support of the ties near their center, were inserted through the lagging. The brick arch, waterproofing, and rock packing were laid up in lifts, in the same manner as in the Twin Tunnel, with grout pipes built in at intervals of about 8 ft. The concrete arch was placed in sections, from 25 to 50 ft. in length, with a rather wet mixture and a back form on the steep slope of the extrados.

The concrete for the sand-walls and lower part of side-walls was handled on tracks and platforms laid on cantilever beams at mid-height of the trestle, as shown by Fig. 3, Plate LXXXII. For the walls above the springing line, the tracks were laid on top of the I-beam ties, and some of the arch concrete, also, was delivered from the mixer at that level and hauled up an incline to the level of the top of the arch. By far the greater part, however, was turned out from mixers set on the completed arch, and was transported on tracks hung in part from the street timbering.

Completion.—Except in the heavily-timbered portions, such as at Fifth Avenue, where the load had to be transferred from posts to the completed masonry section by section, the lining of the tunnels presented no special difficulty. The large number of small forms to be

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set, and the mutual interference of the concreting and duct-laying operations proved to be the most troublesome features of the work.

The restoration of the streets, public utilities, etc., at the open-cut sections was a slow and tedious operation, but the tunnels themselves were completed in March, 1909, 3 years and 10 months after the inception of the work. The finished tunnels are shown by the photograph, Fig. 4, Plate LXXXII, taken at the junction of the twin and three-track types.

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PAPERS AND DISCUSSIONS

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THE NEW YORK TUNNEL EXTENSION OF THE PENNSYLVANIA RAILROAD. THE EAST RIVER TUNNELS.

By James H. Brace, Francis Mason, and S. H. Woodard, Members, Am. Soc. C. E.

To be Presented December 15th, 1909.

This paper will be limited to a consideration of the construction of the tunnels, the broader questions of design, etc., having already been considered in papers by Brig.-Gen. Charles W. Raymond, M. Am. Soc. C. E., and Alfred Noble, Past-President, Am. Soc. C. E.

The location of the section of the work to be considered here is shown on Plate LXV* of Mr. Noble's paper. There are two permanent shafts on each side of the East River and four single cast-iron tube tunnels, each about 6 000 ft. long, and consisting of 3 900 ft. between shafts under the river, and 2 000 ft. in Long Island City, mostly under the depot and passenger yard of the Long Island Railroad. This tube-tunnel work was naturally a single job. The contract for its construction was let to S. Pearson and Son, Incorporated, ground being broken on May 17th, 1904. Five years later, to a day, the work was finished and received its final inspection for acceptance by the Railroad Company.

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

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The contract was of the profit-sharing type, and required an audit, by the Railroad Company, of the contractor's books, and a careful system of cost-keeping by the Company's engineers, so that it is possible to include in the following some of the unit costs of the work. These are given in two parts: The first is called the unit labor cost, and is the cost of the labor in the tunnel directly chargeable to the thing considered. It does not include the labor of operating the plant, nor watchmen, yardmen, pipemen, and electricians. The second is called "top charges," a common term, but meaning different things to different contractors and engineers. Here, it is made to include the cost of the contractor's staff and roving laborers, such as pipemen, electricians, and yardmen, the cost of the plant and its operation, and all miscellaneous expenses, but does not include any contractor's profit, nor cost of materials entering permanent work.

The contractor's plant is to be described in a paper by Henry Japp, M. Am. Soc. C. E., and will not be dealt with here.

The contractors carried on their work from three different sites. From permanent shafts, located near the river in Manhattan, four shields were driven eastward to about the middle of the river; and, from two similar shafts at the river front in Long Island City, four shields were driven westward to meet those from Manhattan. From a temporary shaft, near East Avenue, Long Island City, the land section of about 2 000 ft. was driven to the river shafts.

Tunnels from East Avenue to the River Shafts.

The sinking of the temporary shaft at East Avenue was a fairly simple matter. Rough 6 by 12-in. sheet-piling, forming a rectangle, 127 by 34 ft., braced across by heavy timbering, was driven about 28 ft. to rock as the excavation progressed. Below this, the shaft was sunk into rock, about 27 ft., without timbering. As soon as the shaft was down, on September 30th, 1904, bottom headings were started westward in Tunnels *A*, *B*, and *D*. When these had been driven about half the distance to the river shafts, soft ground was encountered. (See Station 59, Plate LXV.*) As the ground carried considerable water, it was decided to use compressed air. Bulkheads were built in the heading, and, with an air pressure of about 15 lb. per sq. in., the heading was driven through the soft ground and into rock by ordinary

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mining methods. The use of compressed air was then discontinued. West of this soft ground, a top heading, followed by a bench, was driven to the soft ground at about Station 66. Tunnel *C*, being higher, was more in soft ground, and at first it was the intention to delay its excavation until it had been well drained by the bottom headings in the tunnels on each side. A little later it was decided to use a shield without compressed air. This shield had been used in excavating the stations of the Great Northern and City Tunnel in London. It was rebuilt, its diameter being changed from 24 ft. 8½ in. to 23 ft. 5¼ in. It proved too weak, and after it had flattened about 4 in. and had been jacked up three times, the scheme was abandoned, the shield was removed, and work was continued by the methods which were being used in the other tunnels. The shield was rather light, but probably it would have been strong enough had it been used with compressed air, or had the material passed through been all earth. Here, there was a narrow concrete cradle in the bottom, with rock up to about the middle of the tunnel, which was excavated to clear the shield, and gave no support on its sides. The shield was a cylinder crushed between forces applied along the top and bottom.

With the exception of this trial of a shield in Tunnel *C*, and a novel method in Tunnel *B*, where compressed air, but no shield, was used, the description of the work in one tunnel will do for all.

From the bottom headings break-ups were started at several places in each tunnel where there was ample cover of rock above. Where the roof was in soft ground, top headings were driven from the points of break-up and timbered. As soon as the full-sized excavation was completed, the iron lining was built, usually in short lengths.

It will be noticed on Plate LXV* that there is a depression in the rock between Station 65 and the river shafts, leaving all the tunnels in soft ground. As this was directly under the Long Island Railroad passenger station, it was thought best to use a shield and compressed air. This was done in Tunnels *A*, *C*, and *D*, one shield being used successively for all three. It was first erected in Tunnel *D* at Station 64 + 47. From there it was driven westward to the river shaft. It was then taken apart and re-erected in Tunnel *C* at Station 63+63 and driven westward to the shaft. It was then found that there would not be time for one shield to do all four lines. The experience in

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Tunnels *C* and *D* had proven the ground to be much better than had been expected. There was considerable clay in the sand, and, with the water blown out by compressed air, it was very stable. A special timbering method was devised, and Tunnel *B* was driven from Station 66+10 to the shaft with compressed air, but without a shield. In the meantime the shield was re-erected in Tunnel *A* and was shoved through the soft ground from Station 65+48 nearly to the river shaft, where it was dismantled.

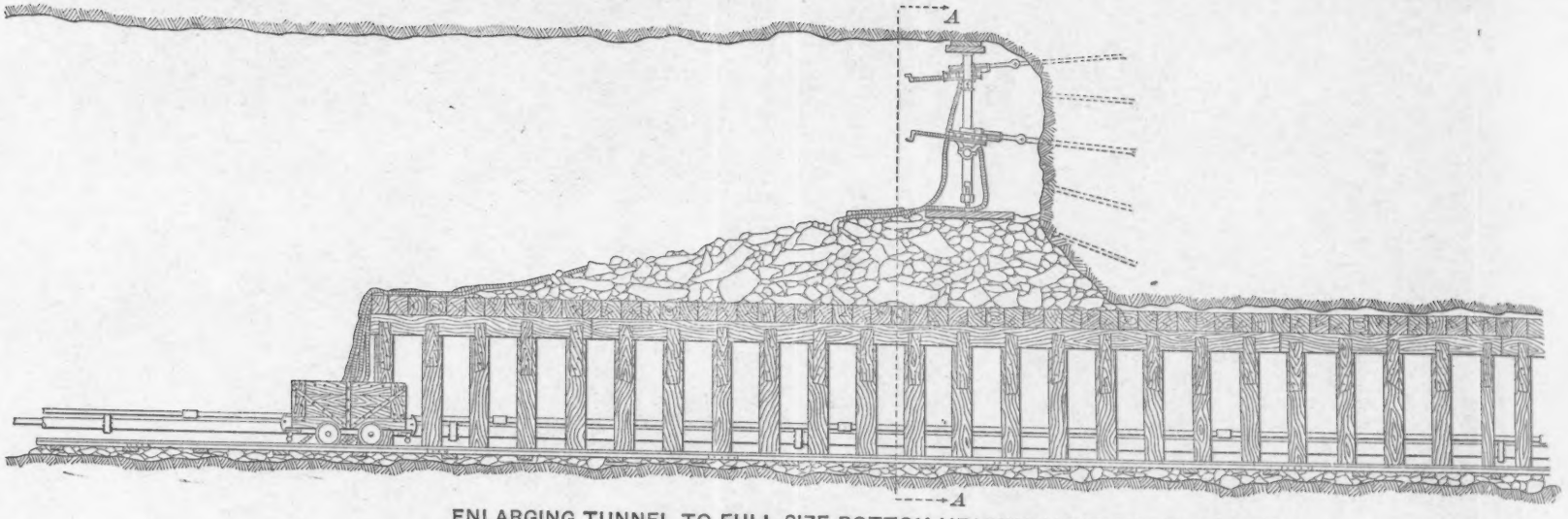
There was nothing unusual about the shield work; it was about the same as that under the river, which is fully described elsewhere. In spite of great care in excavating in front of the shield, and prompt grouting behind it, there was a small settlement of the building above, amounting to about 1½ in. in the walls and about 5 in. in the ground floors which were of concrete laid like a sidewalk directly upon the ground. Whether this settlement was due to ground lost in the shield work or to a compacting of the ground on account of its being dried out by compressed air, it is impossible to say.

The interesting features of this work from East Avenue to the river shafts are the mining methods and the building of the iron tube without a shield.

Excavation In All Rock.

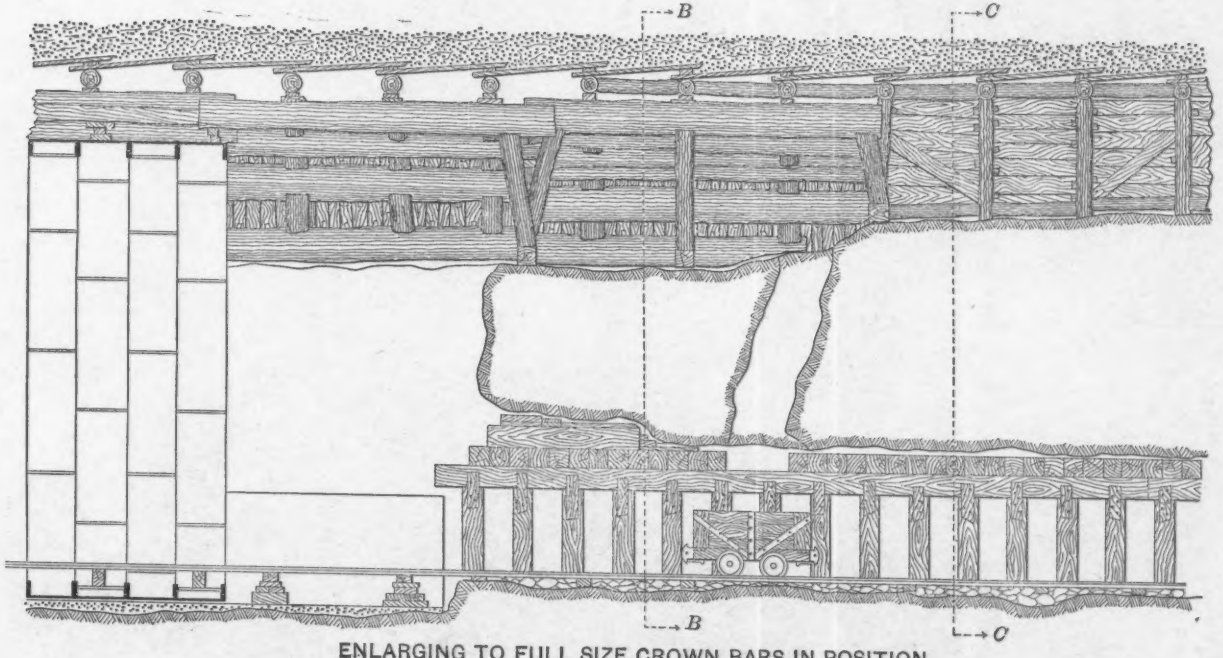
Where the tunnel was all in good rock two distinct methods were used. The first was the bottom-heading-and-break-up, and the second, the top-heading-and-bench method. The first is illustrated by Figs. 1 and 2, Plate LXXXIII. The bottom heading, 13 ft. wide and 9 ft. high, having first been driven, a break-up was started by blasting down the rock, forming a chamber the full height of the tunnel. The timber platform, shown in the drawing, was erected in the bottom heading, and extended through the break-up chamber. The plan was then to drill the entire face above the bottom heading and blast it down upon the timber staging, thus maintaining a passage below for the traffic from the heading and break-ups farther down the line. Starting with the condition indicated by Plate LXV,* the face was drilled, the columns were then taken down and the muck pile was shoveled through holes in the staging into muck cars below. The face was then blasted down upon the staging, the drill columns were set up on the muck pile, and the operation was repeated. This method has the

* *Proceedings*, Am. Soc. C. E., for September, 1909.



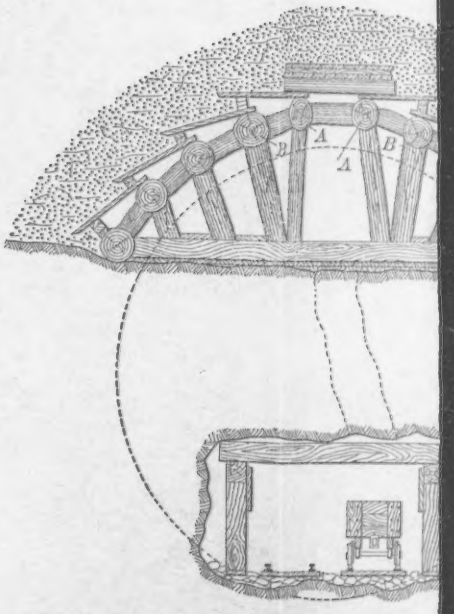
ENLARGING TUNNEL TO FULL SIZE-BOTTOM HEADING HAVING BEEN DRIVEN

FIG.1



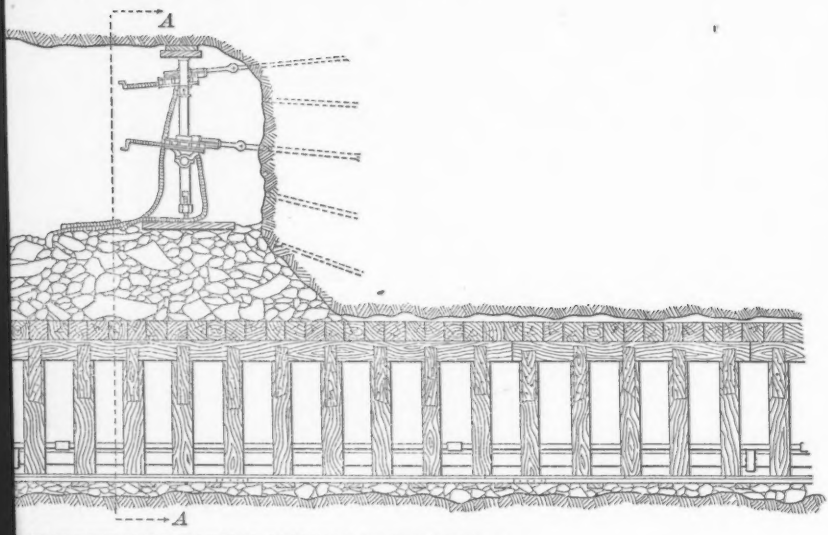
ENLARGING TO FULL SIZE, CROWN BARS IN POSITION

FIG.3



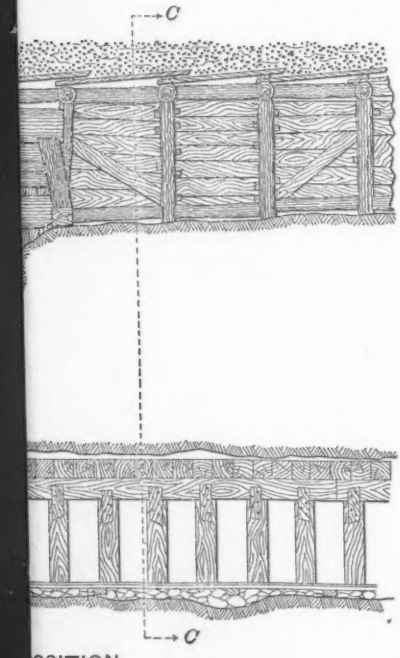
CROSS-SECTION *B-B*

FIG.4

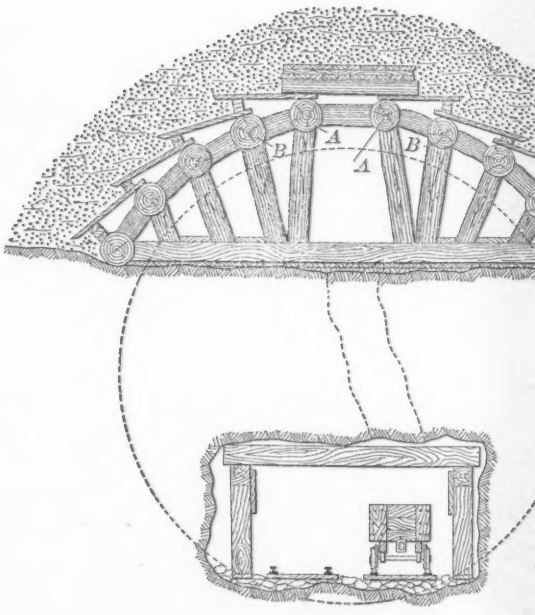


IZE-BOTTOM HEADING HAVING BEEN DRIVEN

FIG.1

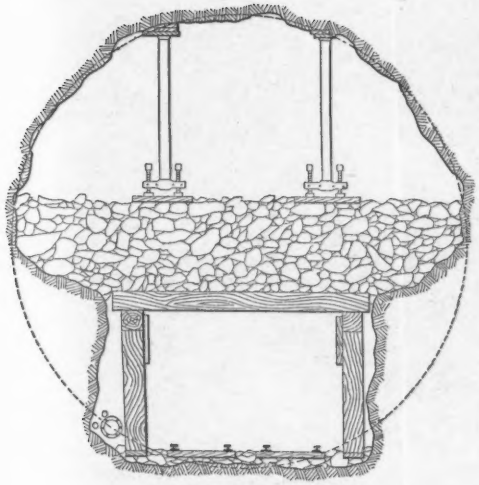


OSITION

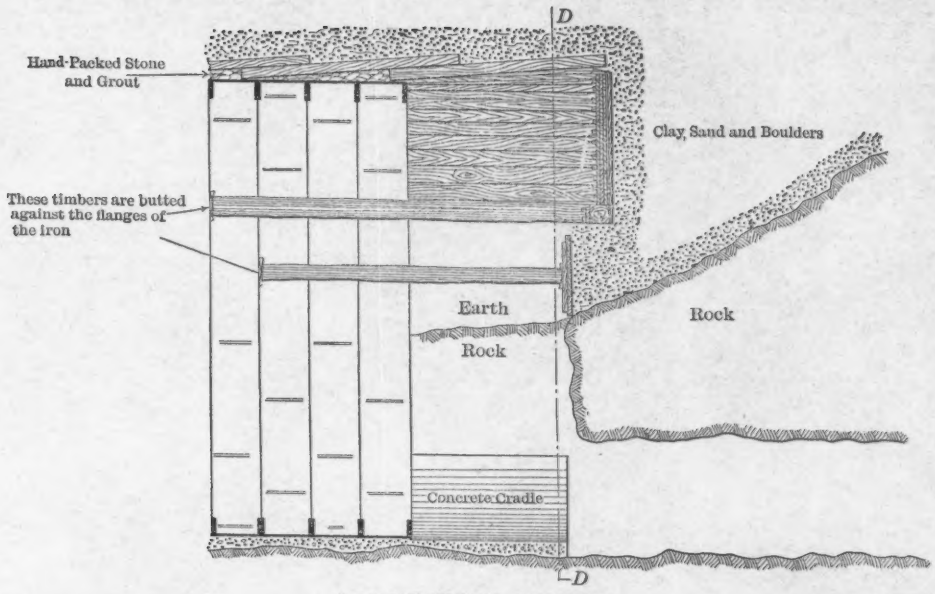


CROSS-SECTION *B-B*

FIG.4

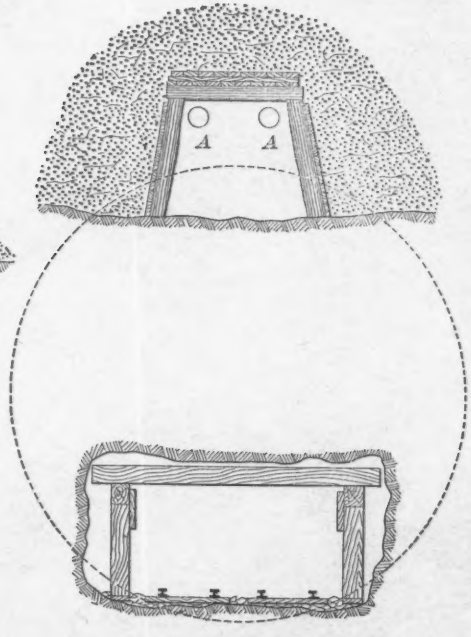


SECTION *A-A*
FIG.2

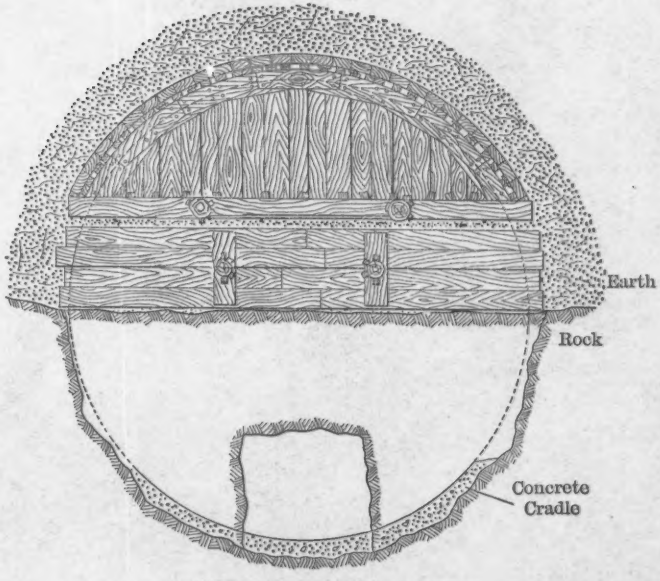


LONGITUDINAL SECTION
FIG.6

ON *B-B*



CROSS-SECTION *C-C*
DRIVING TOP HEADING
FIG.5



CROSS-SECTION *D-D*
FIG.7

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advantage that the bottom heading can be pushed through rapidly, and from it the tunnel may be attacked at a number of points at one time. It was found to be more expensive than the top-heading-and-bench method, and as soon as the depression in the rock at about Station 59 was passed, a top heading about 7 ft. high, and roughly the segment of a 23-ft. circle, was driven to the next soft ground in each of the four tunnels. The remainder of the section was taken out in two benches, the first, about 4 ft. high, was kept about 15 ft. ahead of the lower bench, which was about the remaining 11 ft. high.

Excavation in Earth and Rock.

About 2 500 ft. of tunnel, the roof of which was in soft ground, was excavated in normal air by the mining-and-timbering method. In the greater part of this the rock surface was well above the middle of the tunnel. The method of timbering and mining, while well enough known, has not been generally used in the United States.

Starting from the break-up in all rock, as described above, and illustrated on Plate LXV,* when soft ground was approached, a top heading was driven from the rock into and through the earth. This heading was about 7 ft. high and about 6 ft. wide. This was done by the usual post, cap, and poling-board method. The ground was a running sand with little or no clay, and, at first, considerable water, in places. All headings required side polings. The roof poling boards were about 2½ or 3 ft. above the outside limit of the tunnel lining, as illustrated by Figs. 3, 4, and 5, Plate LXXXIII. The next step was to place two crown-bars, *AA*, usually about 20 ft. long, under the caps. Posts were then placed under the bars, and poling boards at right angles to the axis of the tunnel were then driven out over the bars. As these polings were being driven, the side polings of the original heading were removed, and the earth was mined out to the end of these new transverse polings. Breast boards were set on end under the ends of the transverse polings when they had been driven out to their limit. Side bars, *BB*, were then placed as far out as possible and supported on raking posts. These posts were carried down to rock, if it was near, if not, a sill was placed.

A new set of transverse polings was driven over these side bars and the process was repeated until the sides had been carried down to rock

* *Proceedings*, Am. Soc. C. E., for September, 1909.

or down to the elevation of the sills supporting the posts, which were usually about 4 ft. above the axis of the tunnel.

The plan then was to excavate the remainder of the section and build the iron lining in short lengths, gradually transferring the weight of the roof bars of the iron lining as the posts were taken out. This meant that not more than four rings, and often only one ring, could be built before excavation and a short length of cradle became necessary. Before the posts under the roof bars could be built and the weight transferred to the iron lining, a grout dam was placed at the leading end of the iron lining, and grout was brought up to at least 45° from the top. Such workings were in progress at as many as eight places in one tunnel at the same time. Where there was only the ordinary ground-water to contend with, the driving of the top heading drained the ground very thoroughly, and the enlarging was done easily and without a serious loss of ground. Under these conditions the surface settlement was from 6 in. to 2 ft.

Under Borden Avenue, there was more water, which probably came from a leaky sewer; it was not enough to form a stream, but just kept the ground thoroughly saturated. There was a continued though hardly perceptible flow of earth through every crevice in the timbering during the six or eight weeks between the driving of the top heading and the placing of the iron lining; and here there was a settlement of from 4 to 8 ft. at the surface.

Tunneling in Compressed Air Without a Shield.

When it became evident that there would not be time for one shield to do the soft ground portions of all four tunnels under the Long Island Railroad station, a plan was adopted and used in Tunnel *B* which, while not as rapid, turned out to be as cheap as the work done by the shields. Figs. 6 and 7, Plate LXXXIII, and Fig. 1, Plate LXXXIV, illustrate this work fairly well. The operation of this scheme was about as follows: Having the iron built up to the face of the full-sized excavation, a hole or top heading, about 3 ft. wide and 4 or 5 ft. high, was excavated to about 10 ft. in advance. This was done in a few hours without timbering of any kind; but, as soon as the hole or heading was 10 ft. out, 6 by 12-in. laggings or polings were put up in the roof, with the rear ends resting on the iron lining and the leading ends resting on vertical breast boards. The heading was then widened

FIG. 3.—CUTTING EDGE OF CAISSON ASSEMBLED.

FIG. 4.—CAISSON SUPPORTED ON JACKS AND BLOCKS.

FIG. 1.—TUNNELING IN COMPRESSED AIR WITHOUT SHIELD.

FIG. 3.—CUTTING EDGE OF CAISSON ASSEMBLED.

FIG. 2.—T-HEAD AIR-LOCK.

FIG. 4.—CAISSON SUPPORTED ON JACKS AND BLOCKS.

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out rapidly and the lagging was placed, down to about 45° from the crown. The forward ends of the laggings were then supported by a timber rib and sill. Protected by this roof, the full section was excavated, and three rings of the iron lining were built and grouted, and then the whole process was repeated.

Concrete Cradles, Hand-Packed Stone and Grouting.

Had the East Avenue Tunnel been built by shields, as was contemplated at the time of its design, the space between the limits of excavation and the iron lining would have been somewhat less than by the method actually used, especially in the earth portions. This space would have been filled with grout ejected through the iron lining. The change in the method of doing the work permitted the use of cheaper material, in place of part of the grout, and, at the same time, facilitated the work.

The tube of cast-iron rings is adapted to be built in the tail of the shield. Where no shield was used, after the excavation was completed and all loose rock was removed, timbers were fixed across the tunnel from which semicircular ribs were hung, below which lagging was placed. The space between this and the rough rock surface was filled with concrete. This formed a cradle in which the iron tube could be erected, and, at the same time, occupied space which would have been filled by grout, at greater cost, had a shield been used.

As soon as each ring of iron was erected, the space between it and the roof of the excavation was filled with hand-packed stone. At about every sixth ring a wall of stone laid in mortar was built between the lining and the rock to serve as a dam to retain grout. The interstices between the hand-packed stones were then filled with 1 to 1 grout of cement and sand, ejected through the iron lining. The concrete cradles averaged 1.05 cu. yd. per ft. of tunnel, and cost, exclusive of materials, $6.70 per cu. yd., of which $2.25 was for labor and $4.45 was for top charges. The hand-packed stone averaged 1½ cu. yd. per ft. of tunnel, and cost $2.42 per cu. yd., of which $0.98 was for labor and $1.44 was for top charges.

Erection of Iron Lining.

The contractors planned to erect the iron lining with erectors of the same pattern as that used on the shield under the river, mounted

on a traveling stage. These will be described in detail in Mr. Japp's paper. Two of these stages and erectors worked in each tunnel at different points. The tunnel was attacked from so many points that these erectors could not be moved from working to working. The result was that about 58% of the lining was built by hand. At first thought, this seems to be a crude and extravagant method, as the plates weighed about 1 ton each and about 20 000 were erected by hand. As it turned out, the cost was not greater than for those erected by machinery, taking into account the cost of erectors and power. This, however, was largely because the hand erection reduced the amount of work to be done by the machines so much that the machines had an undue plant charge.

The hand erection was very simple. A portable hand-winch, with a ⅝-in. wire rope, was set in any convenient place. The wire rope was carried to a snatch-block fastened to the top of the iron previously built; or, where the roof was in soft ground, the timbering furnished points of attachment. The end of the wire rope was then hooked to a bolt hole in a new plate, two men at the winch lifted the plate, and three or four others swung it into approximate place, and, with the aid of bars and drift-pins, coaxed it into position and bolted it. Where there was no timbering above the iron, sometimes the key and adjoining plates were set on blocking on a timber staging and then jacked up to place.

Long Island Shafts.

The river shafts were designed to serve both as working shafts and as permanent openings to the tunnels, and were larger and more substantial than would have been required for construction purposes. Plate LXII* of Mr. Noble's paper shows their design. They consist of two steel caissons, each 40 by 74 ft. in plan, with walls 5 ft. thick filled with concrete. A wall 6 ft. thick separated each shaft into two wells 29 by 30 ft., each directly over a tunnel. Circular openings for the tunnel, 25 ft. in diameter, were provided in the sides of the caissons. During the sinking these were closed by bulkheads of steel plates backed by horizontal steel girders. The shafts were sunk as pneumatic caissons to a depth of 78 ft. below mean high water. There have been a few caissons which were larger and were sunk deeper than

* *Proceedings*, Am. Soc. C. E., for September, 1909.

these, but most large caissons have been for foundations, such as bridge piers, and have been stopped at or a little below the surface of the rock. The unusual feature of the caissons for the Long Island shaft is that they were sunk 54 ft. through rock.

It had been hoped that the rock would prove sound enough to permit stopping the caissons at or a little below the surface and continuing the excavation without sinking them further; for this reason only the steel for the lower 40 ft. of the caissons was ordered at first.

The roof of the working chamber was placed 7 ft. above the cutting edge. It was a steel floor, designed by the contractors, and consisted of five steel girders, 6 ft. deep, 29 ft. long, and spaced at 5-ft. centers. Between were plates curved upward to a radius of 4 ft. Each working chamber had two shafts, 3 ft. by 5 ft. in cross-section, with a diaphragm dividing it into two passages, the smaller for men and the larger for muck buckets. On top of these shafts were Moran locks. Mounted on top of the caisson was a 5-ton Wilson crane, which would reach each shaft and also the muck cars standing on tracks on the ground level beside the caissons. Circular steel buckets, 2 ft. 6 in. in diameter and 3 ft. high, were used for handling all muck. These were taken from the bottom of the working chamber, dumped in cars, and returned to the bottom without unhooking. Work was carried on by three 8-hour shifts per day. The earth excavation was done at the rate of about 67 cu. yd. per day from one caisson. The rock excavation, amounting to about 6 200 cu. yd. in each caisson, was done at the rate of about 44.5 cu. yd. per day. The average rate of lowering, when the cutting edge of the south caisson was passing through earth, was 0.7 ft. per day. In rock, the rate was 0.48 ft. per day in the south caisson, and 0.39 ft. per day in the north caisson.

At the beginning all lowering was done with sixteen hydraulic jacks. Temporary brackets were fastened to the outside of the caisson. A 100-ton hydraulic jack was placed under each alternate bracket and under each of the others there was blocking. The jacks were connected to a high-pressure pump in the power-house. As the jacks lifted the caisson, the blocking was set for a lower position, to which the caisson settled as the jacks were exhausted. After the caisson had penetrated the earth about 10 ft., the outside brackets were removed and the lowering was regulated by blocking placed under brackets in the working chamber. The caisson usually rested on

three sets of blockings on each side and two on each end. The blocking was about 4 ft. inside the cutting edge. In the rock, as the cutting edge was cleared for a lowering of about 2 ft., 6 by 8-in. oak posts were placed under the cutting-edge angle. When a sufficient number of posts had been placed, the blocking on which the caisson had rested was knocked or blasted out, and the rock underneath was excavated. The blocking was then re-set at a lower elevation. The posts under the cutting edge were then chopped part way through and the air pressure was lowered about 10 lb., which increased the net weight to more than 4 000 000 lb. The posts then gradually crushed and the caissons settled to the new blocking. The tilt or level of the caisson was controlled by chopping the posts more on the side which was desired to move first.

The caisson nearly always carried a very large net weight, usually about 870 tons. The concrete in the walls, which was added as the caisson was being sunk, was kept at about the elevation of the ground. There was generally a depth of from 5 to 20 ft. of water ballast on top of the roof of the working chamber. The air pressure in the working chamber was usually much less than the hydrostatic head outside the caisson. For example, the average air pressure in the south caisson during January, 1906, was 16½ lb., while the average head was 62.5 ft., equivalent to 27 lb. per sq. in. Under these conditions, there was a continued but small leakage into the caisson of from 15 000 to 20 000 gal. per day.

In the rock the excavation was always carried from 2 to 5 in. outside the cutting edge. As soon as the cutting edge was cleared, bags of clay were placed under it in a well-tiered, solid pile, so that when the caisson was lowered the bags were cut through and most of the clay, bags and all, was squeezed back of the cutting edge between the rock and the caisson.

Table 1 shows the relation of the final position of the caissons to that designed.

The cost of rock excavation in the caisson was $4.48 per cu. yd. for labor and $10.54 for top charges.

The bottom of the shaft is an inverted concrete arch, 4 ft. thick, water-proofed with 6-ply felt and pitch. As soon as the caisson was down to its final position and the excavation was completed, concrete was deposited on the uneven rock surfaces, brought up to the line of the water-proofing, and given a smooth 1-in. mortar coat. The felt

TABLE 1.—RELATION OF THE FINAL POSITION OF THE CAISSONS TO THAT DESIGNED.

LOCATION.	LONG ISLAND CITY.						MANHATTAN.					
Shaft.	North.			South.			North.			South.		
Corner.	High.	East.	North.	High.	East.	North.	High.	East.	South.	High.	East or West.	North or South.
Northeast...	0.21 ft.	0.08 ft.	0.05 ft.	0.32 ft.	0.15 ft.	0.28 ft.	0.28 ft.	0.74 ft.	0.38 ft.	0.00ft.	0.06 ft. east.	0.04 ft. south.
Northwest...	0.22 "	0.08 "	0.02 "	0.00 "	0.15 "	0.12 "	0.00 "	0.74 "	0.22 "	0.08 "	0.06 " "	0.13 " north.
Southwest...	0.27 "	0.14 "	0.02 "	0.18 "	0.45 "	0.12 "	0.11 "	0.31 "	0.22 "	0.21 "	0.45 " west.	0.13 " "
Southeast...	0.22 "	0.14 "	0.05 "	0.39 "	0.45 "	0.28 "	0.46 "	0.31 "	0.38 "	0.04 "	0.45 " "	0.04 " south.

was stuck together in 3-ply mats on the surface with hot coal-tar pitch. These were rolled and sent down into the working chamber, where they were put down with cold pitch liquid at 60° Fahr. Each sheet of felt overlapped the one below 6 in. The water-proofing was covered by a 1-in. mortar plaster coat, after which the concrete of the 4-ft. inverted arch was placed. While the water-proofing and concreting were being done, the air pressure was kept at from 30 to 33 lb. per sq. in., the full hydrostatic head at the cutting edge. After standing for ten days, the air pressure was taken off, and the removal of the roof of the working chamber was begun. The water-proofing was done by the Union Construction and Waterproofing Company.

The cost of labor in compressed air chargeable to concreting was $3.40 per cu. yd.

After the roof of each working chamber had been removed, the shield was erected on a timber cradle in the bottom of the shaft, in position to be shoved out of the opening in the west side of the caisson. Temporary rings of iron lining were erected across the shaft in order to furnish something for the shield jacks to shove against.

The roof of the working chamber was then re-erected about 35 ft. above its original position and about 8 ft. above the tunnel openings. This time, instead of the two small shafts which were in use during the sinking of the caisson, a large steel shaft with a T-head lock was built. This is illustrated in Fig. 2, Plate LXXXIV. The shaft was 8 ft. in diameter. Inside there was a ladder and an elevator cage for lowering and hoisting men and the standard 1-yd. tunnel cars. At the top, forming the head of the T, there were two standard tunnel locks.

Manhattan Shafts.

A permanent shaft, similar to the river shafts in Long Island City, was constructed at Manhattan over each pair of tunnels. Each shaft was located across two lines, with its longer axis transverse to the tunnels. Plate LXV* shows their relative positions. They were divided equally by a reinforced concrete partition wall transverse to the line of the tunnels. On completion, the western portions were turned over to the contractor for the cross-town tunnels for his exclusive use.

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* *Proceedings*, Am. Soc. C. E., for September, 1909.

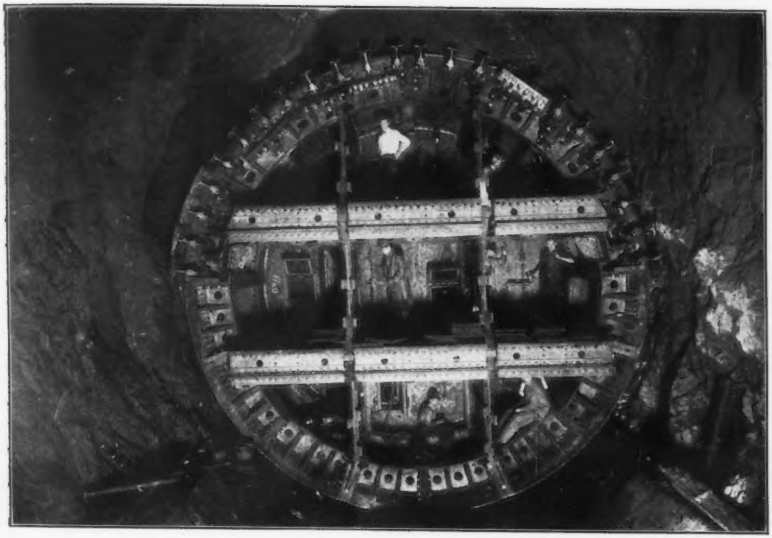

FIG. 1.—SHIELD FITTED WITH SECTIONAL SLIDING HOODS AND SLIDING EXTENSIONS TO THE FLOORS.

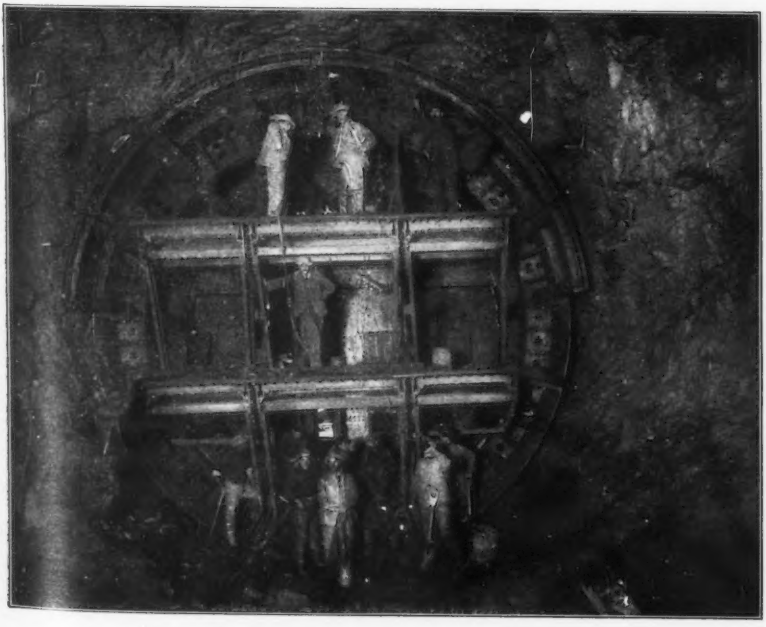

FIG. 2.—SHIELD FITTED WITH FIXED HOODS AND FIXED EXTENSIONS TO THE FLOORS.

South Shaft.—Work on the south shaft was started on June 9th, 1904, with the sinking of a 16 by 16-ft. test pit in the center of the south half of the south shaft, which reached disintegrated rock at a depth of about 20 ft.

Starting in August, the full shaft area, 74 by 40 ft., was taken out in an open untimbered cut to the rock, and a 20 by 50-ft. shaft was sunk through the rock to tunnel grade, leaving a 10 or 12-ft. berm around it. (Fig. 1, Plate XC.)

The erection of the caisson was started, about the middle of January, on the rock berm surrounding the 20 by 50-ft. shaft and about 15 ft. below the surface. Fig. 3, Plate LXXXIV, shows the cutting edge of the caisson assembled. The excavation of the small shaft had shown that hard rock and only a very small quantity of water would be encountered, and that the caisson need be sunk only a short distance below the rock surface. Therefore, no working-chamber roof was provided, the caisson was built to a height of only 40 ft., and the circular openings were permanently closed.

The assembling of the caisson took 2½ months, and on April 2d lowering was started. Inverted brackets were bolted temporarily to the cutting-edge stiffening brackets, and the sinking was carried on by methods similar to those used at Long Island. The jacks and blocking supporting the caisson are shown in Fig. 4, Plate LXXXIV. As soon as the cutting edge entered the rock, which was drilled about 6 in. outside of the neat lines, the space surrounding the caisson was back-filled with clay and muck to steady it and provide skin friction. As the friction increased, the walls were filled with concrete, and as the caisson slowly settled, it was checked and guided by blocking. The cutting edge finally came to rest 31 ft. below mean high water, the sinking having been accomplished in about seven weeks, at an average rate of 0.50 ft. per day.

The final position of the cutting edge in relation to its designed position is shown in Table 1.

A berm about 4 ft. wide was left at the foot of the caisson below which the rock was somewhat fissured and required timbering. The cutting edge of the caisson was sealed to the rock with grout on the outside and a concrete base to the caisson walls on the inside, the latter resting on the 4-ft. berm. Following the completion of the shaft, the permanent sump was excavated to grade for use during construction.

North Shaft.—The north shaft had to be sunk in a very restricted area. The east side of the caisson cleared an adjoining building at one point by only 1 ft., while the northwest corner was within the same distance of the east line of First Avenue. As in the case of the Long Island shafts, the steelwork for only the lower 40 ft. was ordered at the start. This height was completely assembled before sinking was begun. The caisson was lowered in about the same manner as those previously described. The bearing brackets for the hydraulic jacks were attached, as at the south shaft, to the inside of the cutting-edge brackets. The east side of the caisson was in contact with the foundations of the neighboring building, while the west side was in much softer material. As a consequence, the west side tended to settle more rapidly and thus throw the caisson out of level and position. To counteract that tendency, it was necessary to load the east wall heavily with cast-iron tunnel sections, in addition to the concrete filling in the walls.

Soon after sinking was begun, a small test shaft was sunk to a point below the elevation of the top of the tunnels. The rock was found to be sound, hard, and nearly dry. It was then decided to stop the caisson as soon as a foundation could be secured on sound rock. The latter was found at a depth of 38 ft. below mean high water. With the cutting edge seated at that depth, the top of the caisson was only 2 ft. above mean high water, and as this was insufficient protection against high tides, a 10-ft. extension was ordered for the top. Work, however, went on without delay on the remainder of the excavation. The junction between the cutting edge and the rock was sealed with concrete and grout. The caisson was lowered at an average rate of 0.53 ft. per day. The size of the shaft below the cutting edge was 62 ft. 7 in. by 32 ft. The average rate of excavation during the sinking in soft material was 84 cu. yd. per day. The average rate of rock excavation below the final position of the cutting edge was 125 cu. yd. per day. There were night and day shifts, each working 10 hours. Excavation in earth cost $3.96 per cu. yd., of which $1.45 was for labor and $2.51 for top charges, etc. The excavation of rock cost $8.93 per cu. yd., $2.83 being for labor and $6.10 for top charges.

The final elevations of the four corners of the cutting edge, together with their displacement from the desired positions, are shown in Table 1.

River Tunnels.

The four river tunnels, between the Manhattan and Long Island City shafts, a distance of about 3 900 ft., were constructed by the shield method. Eight shields were erected, one on each line in each shaft, the four from Manhattan working eastward to a junction near the middle of the river with the four working westward from Long Island City. Toward the end of the work it was evident that the shields in Tunnels *B*, *C*, and *D* would meet in the soft material a short distance east of the Blackwell's Island Reef if work were continued in all headings. In order that the junction might be made in firm material, work from Manhattan in those three tunnels was suspended when the shields reached the edge of the ledge. The shields in Tunnel *A* met at a corresponding point without the suspension of work in either. An average of 1 760 ft. of tunnel was driven from Manhattan and 2 142 ft. from Long Island City.

Tunnels Driven Eastward from Manhattan.

Materials and Inception of Work.—The materials encountered are shown in the profile on Plate LXV,* and were similar in all the tunnels. In general, they were found to be about as indicated in the preliminary borings. The materials met in Tunnel *A* may be taken as typical of all.

From the Manhattan shaft eastward, in succession, there were 123 ft. of all-rock section, 87 ft. of part earth and part rock, 723 ft. of all earth, 515 ft. of part rock and part earth, 291 ft. of all rock, and 56 ft. of part rock and part earth.

The rock on the Manhattan side was Hudson schist, while that in the reef was Fordham gneiss. Here, as elsewhere, they resembled each other closely; the gneiss was slightly the harder, but both were badly seamed and fissured. Wherever it was encountered in this work, the rock surface was covered by a deposit of boulders, gravel, and sand, varying in thickness from 4 to 10 ft. and averaging about 6 ft.

The slope of the surface of the ledge on the Manhattan side averaged about 1 vertical to 4 horizontal. The rock near the surface was full of disintegrated seams, and was badly broken up. It was irregularly stratified, and dipped toward the west at an angle of about

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60 degrees. Large pieces frequently broke from the face and slid into the shield, often exposing the sand. The rock surface was very irregular, and was covered with boulders and detached masses of rock embedded in coarse sand and gravel. The sand and gravel allowed the air to escape freely. By the time the shields had entirely cleared the rock, the material in the face had changed to a fine sand, stratified every few inches by very thin layers of chocolate-colored clayey material. This is the material elsewhere referred to as quicksand. As the shield advanced eastward, the number and thickness of the layers of clay increased until the clay formed at least 20% of the entire mass, and many of the layers were 2 in. thick.

At a distance of about 440 ft. beyond the Manhattan ledge, the material at the bottom of the face changed suddenly to one in which the layers of clay composed probably 98% of the whole. The sand layers were not more than $\frac{1}{16}$ in. thick and averaged about 2 in. apart. The surface of the clay rose gradually for a distance of 40 ft. in Tunnels *A* and *B*, and 100 ft. in Tunnels *C* and *D*, when gravel and boulders appeared at the bottom of the shield. At that time the clay composed about one-half of the face.

The surfaces of both the clay and gravel were irregular, but they rose gradually. After rock was encountered, the formations of gravel and clay were roughly parallel to the rock surface.

As the surface of the rock rose they disappeared in order and were again encountered when the shields broke out of rock on the east side of the Blackwell's Island Reef. East of the reef a large quantity of coarse open sand was present in the gravel formations before the clay appeared below the top of the cutting edge. In Tunnels *C* and *D* this was especially difficult to handle. It appears to be a reasonable assumption that the layer of clay was continuous across the reef. Wherever the clay extended above the top of the shield it reduced the escape of air materially. It is doubtless largely due to this circumstance that the part-rock sections in the reef were not the most difficult portions of the work.

While sinking the lower portions of the shafts the tunnels were excavated eastward in the solid rock for a distance of about 60 ft., where the rock at the top was found to be somewhat disintegrated. This was as far as it was considered prudent to go with the full-sized section without air pressure. At about the same time top headings

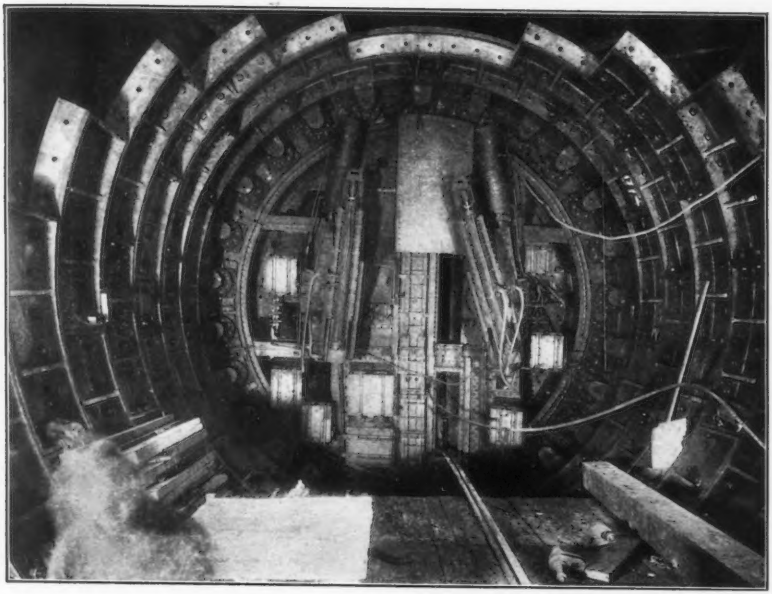

FIG. 1.—REAR OF SHIELD SHOWING COMPLETE FITTINGS.

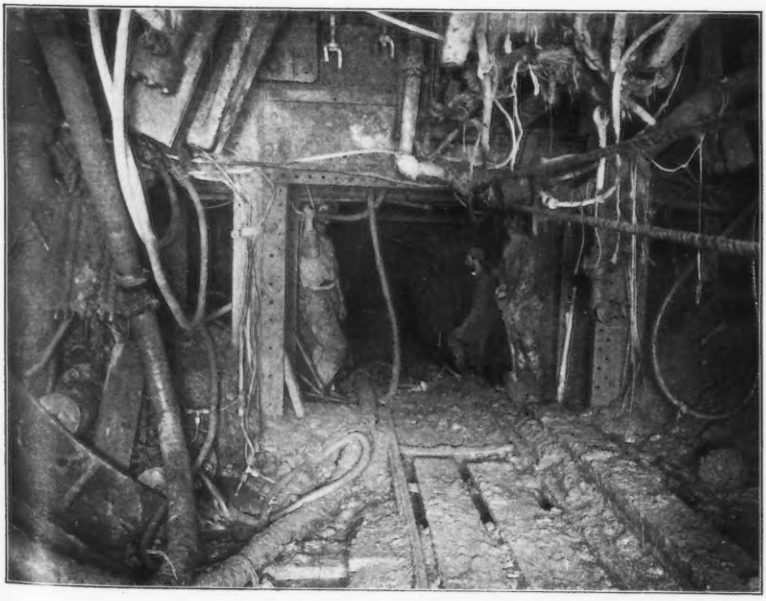

FIG. 2.—SHIELD WITH LOWER PORTION OF BULKHEAD REMOVED.

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were excavated westward from the shafts for a distance of 100 ft., and the headings were enlarged to full size for 50 ft. The object was to avoid damage to the shaft and interference with the river tunnel when work was started by the contractor for the cross-town tunnel.

The shields were erected on timber cradles in the shaft, and were shoved forward to the face of the excavation. Concrete bulkheads, with the necessary air-locks, were then built across the tunnels behind the shields. The shields were erected before the dividing walls between the two contracts were placed. Rings of iron tunnel lining, backed by timbers spanning the openings on the west side, were erected temporarily across the shafts in order to afford a bearing for the shield jacks while shoving into the portals. The movement of the shield eastward was continued in each tunnel for a distance of about 60 ft., and the permanent cast-iron tunnel lining was erected as the shield advanced. Before breaking out of rock, it was necessary to have air pressure in the tunnels. This required the building of bulkheads with air-locks inside the cast-iron linings just east of the portals. Before erecting the bulkheads it was necessary to close the annular space between the iron tunnel lining and the rock. The space at the portal was filled with a concrete wall. After about twenty permanent rings had been erected in each tunnel, two rings were pulled apart at the tail of the shield and a second masonry wall or dam was built. The space between the two dams was then filled with grout. To avoid the possibility of pushing the iron backward after the air pressure was on, rings of segmental plates, $\frac{5}{8}$ in. thick and $13\frac{7}{8}$ in. wide, were inserted in eighteen circumferential joints in each tunnel between the rings as they were erected. The plates contained slotted holes to match those in the segments. After the rings left the shield, the plates were driven outward, and projected about 5 in. When the tunnel was grouted, the plates were embedded.

The bulkheads were completed, and the tunnels were put under air pressure on the following dates:

Line *D*, on October 5th, 1905;
Line *C*, on November 6th, 1905;
Line *B*, on November 25th, 1905;
Line *A*, on December 1st, 1905.

This marked the end of the preparatory period.

In the deepest part of the river, near the pier-head line on the

Manhattan side, there was only 8 ft. of natural cover over the tops of the tunnels. This cover consisted of the fine sand previously described, and it was certain that the air would escape freely from the tunnels through it. To give a greater depth of cover and to check the loss of air, the contractor prepared to cover the lines of the tunnels with blankets of clay, which, however, had been provided for in the specifications. Permits, as described later, were obtained at different times from the Secretary of War, for dumping clay in varying thicknesses over the line of work. The dumping for the blanket allowed under the first permit was completed in February, 1906. The thickness of this blanket varied considerably, but averaged 10 or 12 ft. on the Manhattan side. The original blanket was of material advantage, but the depth of clay was insufficient to stop the loss of air.

The essential parts of the shields in the four tunnels were exactly alike. Those in Tunnels *B* and *D*, however, were originally fitted with sectional sliding hoods and sliding extensions to the floors of the working chambers, as shown by Fig. 1, Plate LXXXV. The shields in Tunnels *A* and *C* were originally fitted with fixed hoods and fixed extensions to the floors, as shown in Fig. 2, Plate LXXXV. A full description of the shields will be found in Mr. Japp's paper.

The shields in each pair of tunnels were advanced through the solid rock section about abreast of each other, until test holes from the faces indicated soft ground within a few feet. As the distance between the sides of the two tunnels was only 14 ft., it was thought best to let Tunnels *B* and *D* gain a lead of about 100 ft. before Tunnels *A* and *C* opened out into soft ground, in order that a blow from one tunnel might not extend to the other. Work in Tunnel *C* was shut down on December 23d, 1905, after exposing sand to a depth of 3 ft. at the top, and it remained closed for seven weeks. Work in Tunnel *A* was suspended on September 29th, 1905. By the time Tunnel *B* had made the required advance, it, together with Tunnels *C* and *D* was overtaxing the capacities of the compressor plant. Only a little work was done in Tunnel *C* until July, 1906, and work in Tunnel *A* was not resumed until October 22d, 1906.

Tunnels Driven Westward from Long Island City.

Materials and Inception of the Work.—The materials met in Tunnel *A* are typical of all four tunnels. From the Long Island shafts

PLATE LXXXVII.
PAPERS, AM. SOC. C. E.
OCTOBER, 1909.
BRACE, MASON, AND WOODARD ON
EAST RIVER TUNNELS: PENNSYLVANIA RAILROAD.

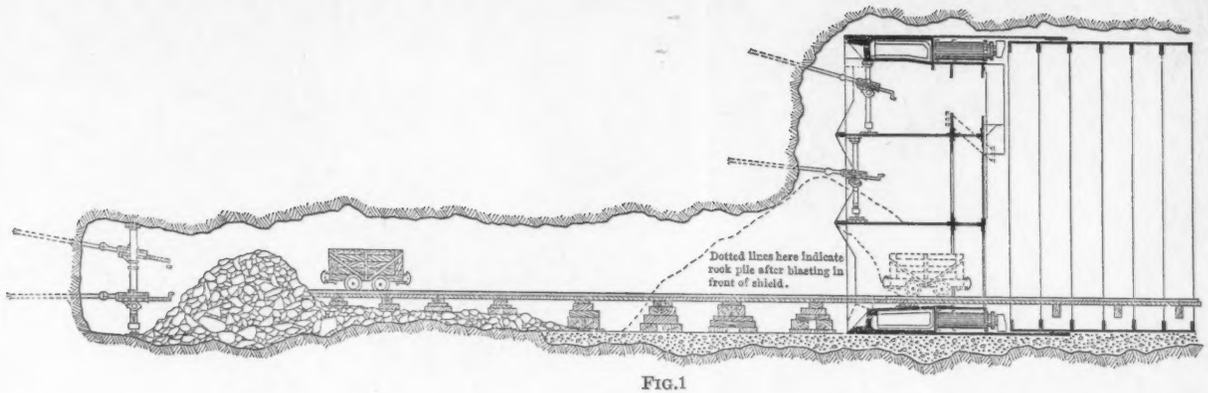
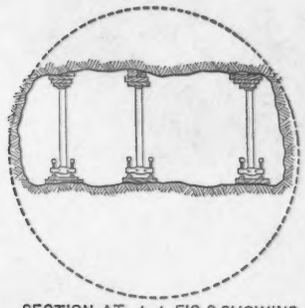


FIG.1



SECTION AT *A-A*, FIG.3 SHOWING DRILL COLUMNS

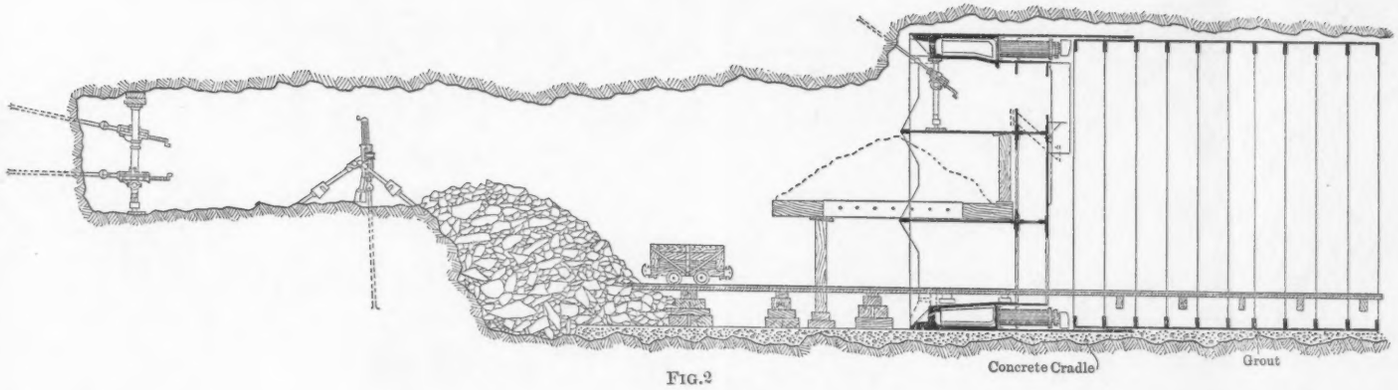
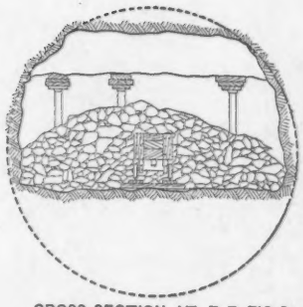


FIG.2



CROSS-SECTION AT *B-B*, FIG.3

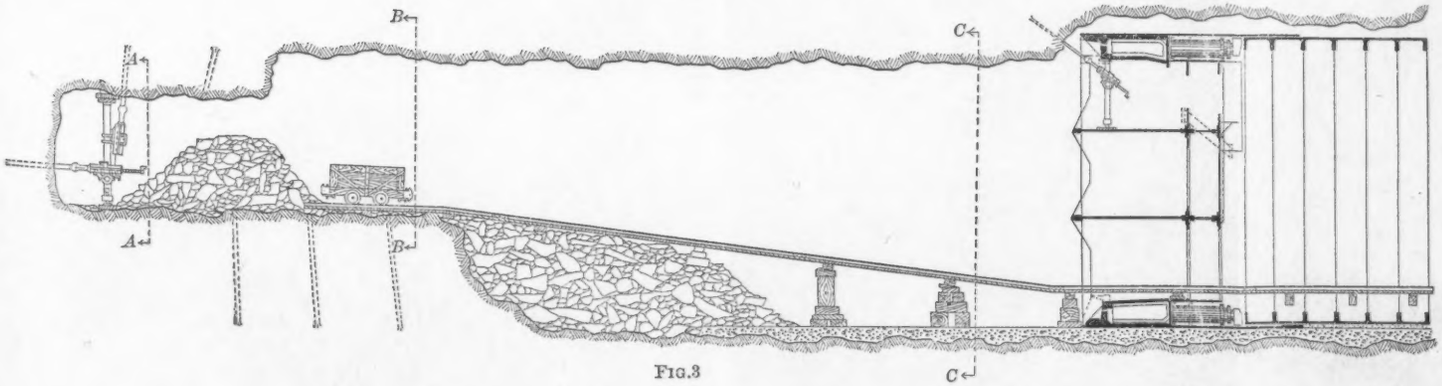
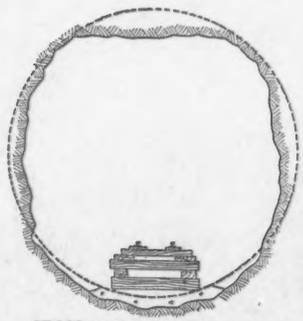


FIG.3



CROSS-SECTION AT *C-C*, FIG.3

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westward, in succession, there were 124 ft. of all-rock section, 125 ft. of part rock and part earth section, 22 ft. of all-rock section, 56 ft. of part rock and part earth section, 387 ft. of all-rock section, 70 ft. of part earth and part rock section, and 1 333 ft. of all-earth section.

The materials passed through are indicated on Plate LXV.* The rock was similar to that of the Blackwell's Island Reef, and was likewise covered by a layer of sand and boulders. The remainder of the soft ground was divided into three classes. The first was a very fine red sand, which occurred in a layer varying in thickness from 6 ft. to at least 15 ft. It may have been much deeper above the tunnel. It is the quicksand usually encountered in all deep foundations in New York City. The following is the result of the sifting test of this sand:

		Held on	No. 30	sieve	0.6%
Passed	No. 30,	"	" No. 40	"	0.4%
"	No. 40,	"	" No. 50	"	0.7%
"	No. 50,	"	" No. 60	"	2.4%
"	No. 60,	"	" No. 80	"	14.9%
"	No. 80,	"	" No. 100	"	54.0%
"	No. 100,	"	" No. 200	"	8.0%
"	No. 200			"	19.0%
					100.0%

This means that grains of all but 4% of it were less than 0.0071 in. in diameter. The 19% which passed the No. 200 sieve, the grains of which were 0.0026 in. or less in diameter, when observed with a microscope appeared to be perfectly clean grains of quartz; to the eye it looked like ordinary building sand, sharp, and well graded from large to small grains. This sand, with a surplus of water, was quick. With the water blown out of it by air pressure, it is stable, stands up well, and is very easy to work. It appears to be the same as the reddish quicksand found in most deep excavations around New York City.

The second material was pronounced "bull's liver" by the miners as soon as it was uncovered. "Bull's liver" seems to be a common term among English-speaking miners the world over. It is doubtful, however, if it is always applied to the same thing. In this case it consisted of layers of blue clay and very fine red sand. The clay seemed to be perfectly pure and entirely free from sand. It would break

* *Proceedings*, Am. Soc. C. E., for September, 1909.

easily with a clean, almost crystalline, fracture, and yet it was soft and would work up easily. The layers of clay varied in thickness from $\frac{1}{16}$ in. to 1 in., while the thickness of the sand layer varied from $\frac{1}{4}$ in. to several inches. The sand was the same as the quicksand already described.

The "bull's liver" was ideal material in which to work a shield. It stood up as well and held the air about as well as clay, and was much easier to handle.

The third material was a layer of fine gray sand which was encountered in the top of all the tunnels for about 400 ft. just east of Blackwell's Island Reef. It was very open, and had grains of rather uniform size.

During the starting out of the tunnels from the shafts, and for more than a year afterward, the roof of the working chamber in the caissons and the locks previously described under the Long Island shafts took the place of the bulkhead across the tunnels for confining the air pressure.

The first work in air pressure was to remove the shield plug closing the opening in the side of the shaft. This being done, the shield was shoved through the opening, and excavation begun.

At the start the shields were fitted with movable platforms, but no hoods of any kind were placed until after the rock excavation was completed.

Methods of Excavation.

The distribution of materials to be excavated, as previously outlined, divided the excavation into three distinct classes, for which different methods had to be developed.

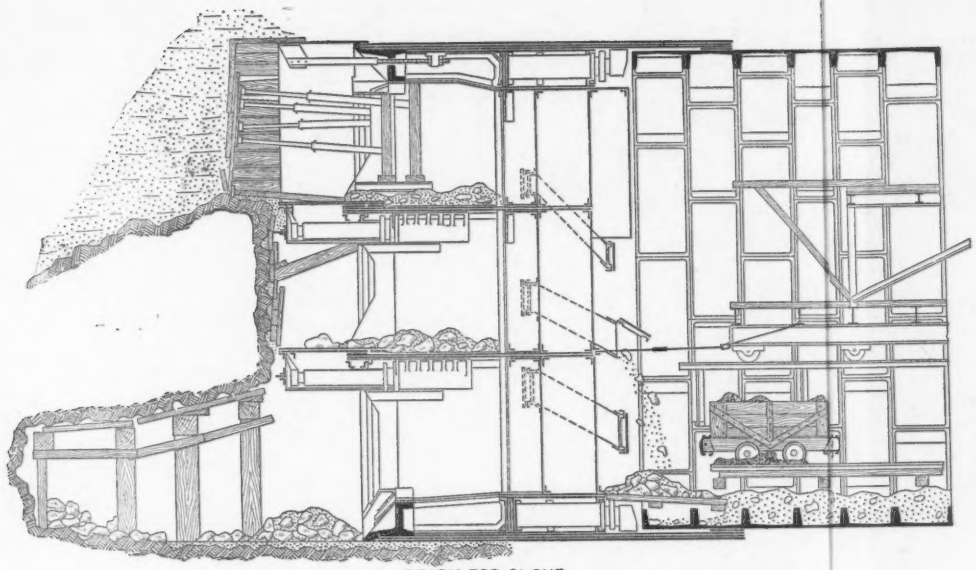
These three classes were:

First.—All-rock section.

Second.—Rock in the bottom, earth in the top.

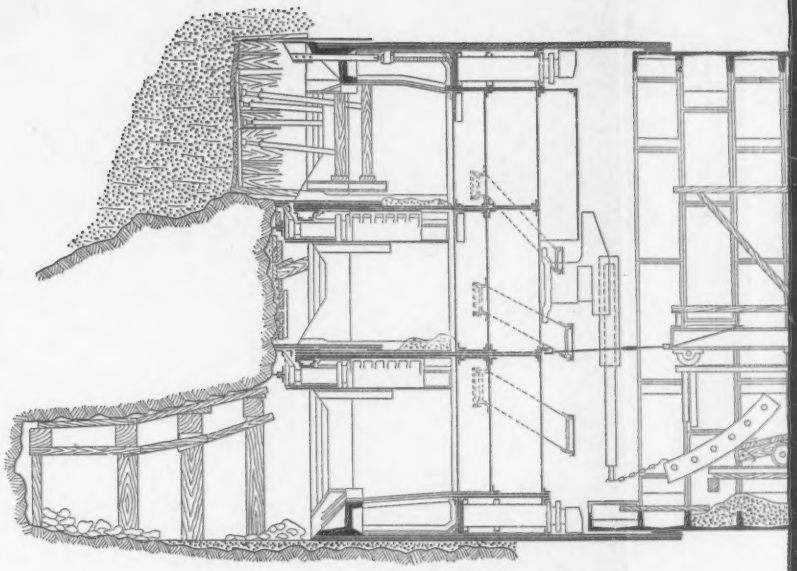
Third.—All-earth section.

The extent of the second and third classes was much greater than that of the first, and they, of course, determined the use of the shield. Shields had not previously been used extensively in rock work, either where the face was wholly or partly in rock, and it was necessary to develop the methods by experience. The specifications required that where rock was present in the bottom, a bed of concrete should be laid in the form of a cradle on which to advance the shield.



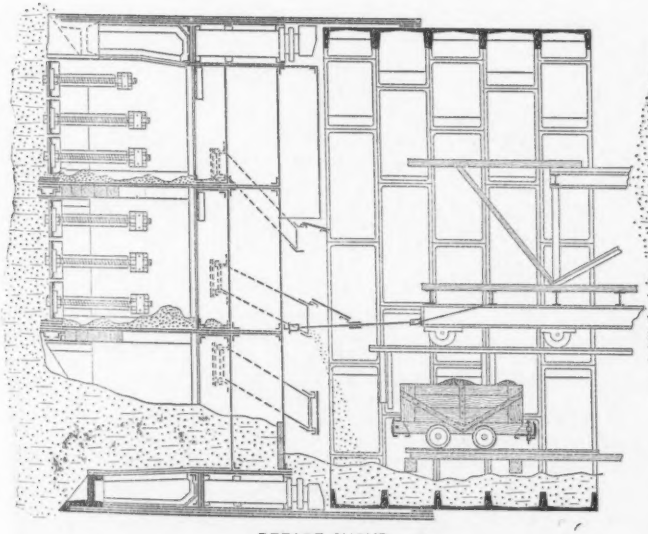
READY FOR SHOVE

FIG. 1.



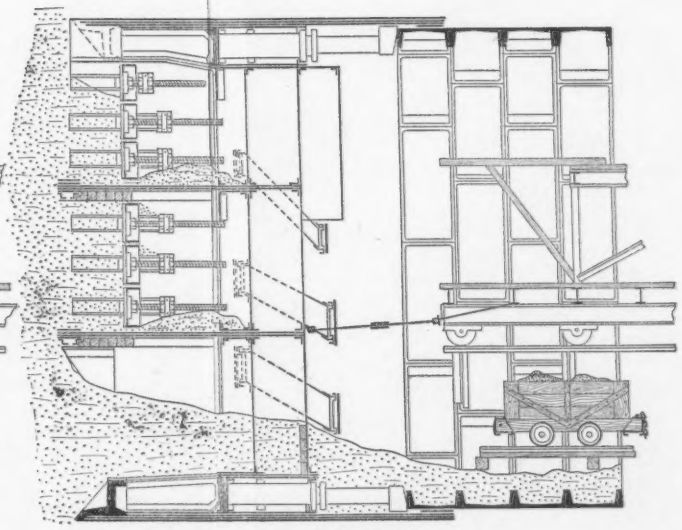
AFTER SHOVE, ERECTING IRON

FIG.2



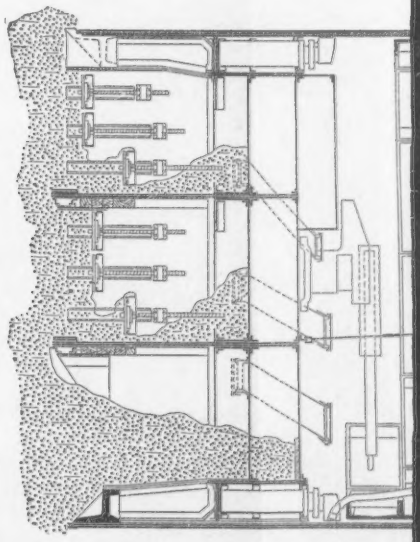
BEFORE SHOVE

FIG. 4.



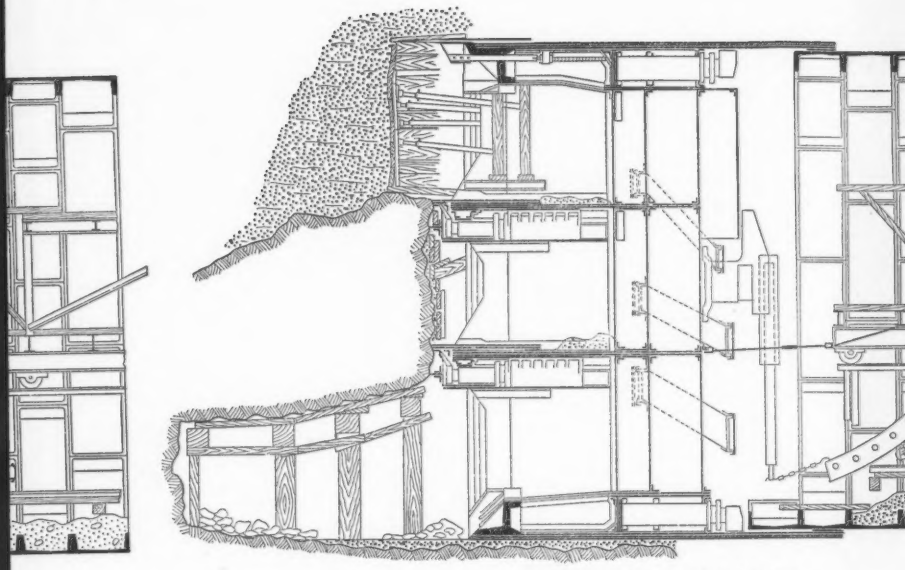
AFTER SHOVE

FIG. 5.



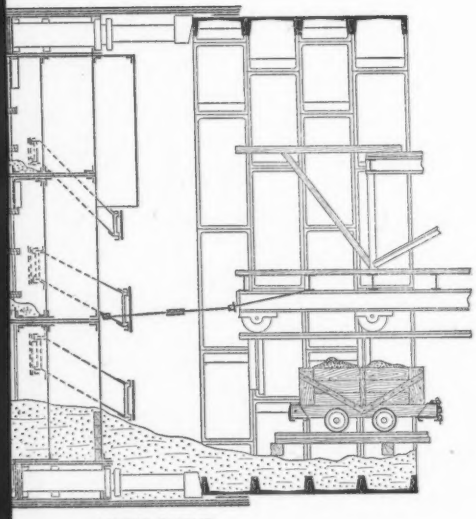
PREPARING FOR 2D SHOVE AND PLACIN

FIG.6



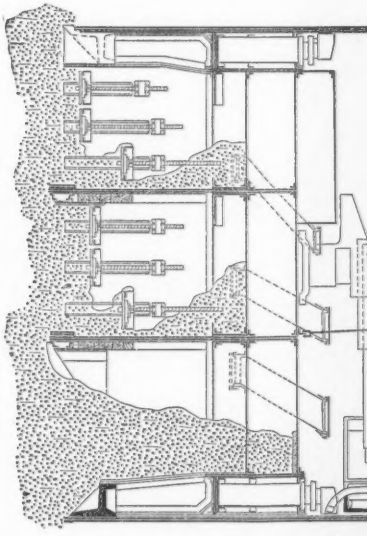
AFTER SHOVE, ERECTING IRON

FIG.2



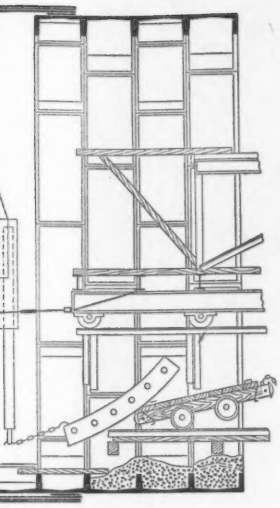
AFTER SHOVE

FIG. 5.



PREPARING FOR 2D SHOVE AND PL

FIG.6



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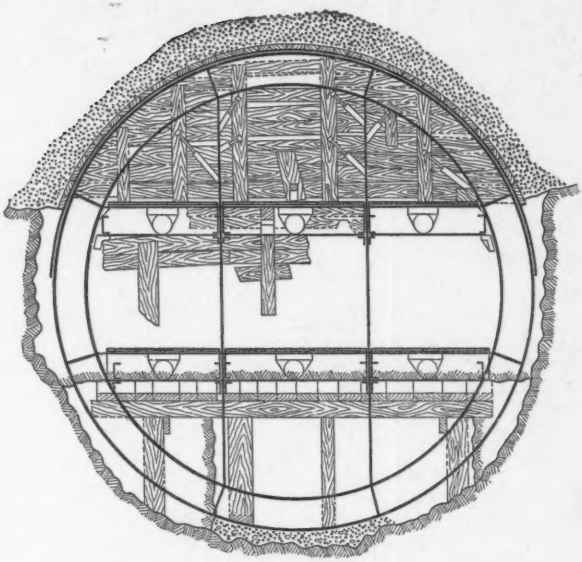
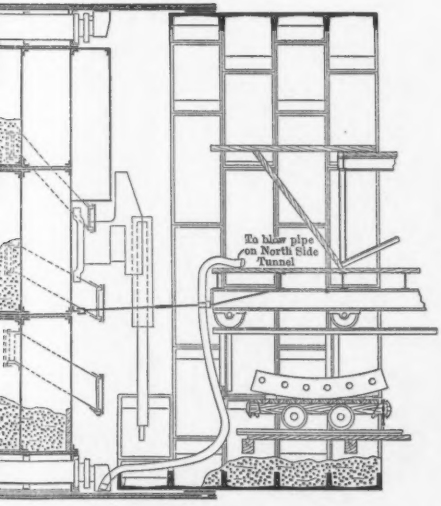
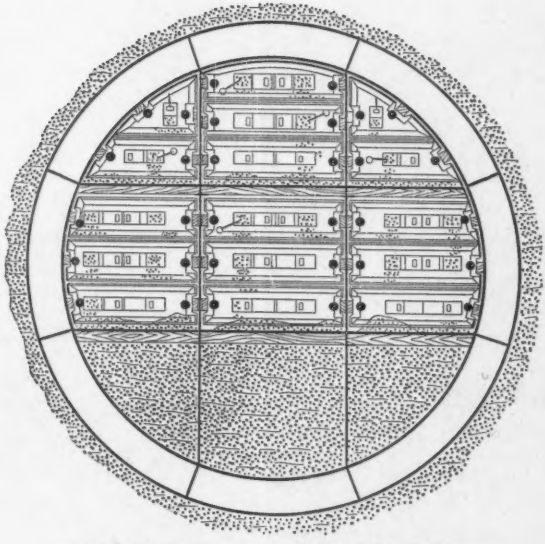


SECTION SHOWING BREASTING
FIG.3



D SHOVE AND PLACING BOTTOM PLATES
FIG.6



BACK ELEVATION OF SHUTTERS, SHOWING SLIDES
FIG.7

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All Rock.—At different times, three general methods were used for excavating in all-rock sections. They may be called: The bottom-heading method; the full-face method; and the center-heading method.

The bottom-heading method was first tried. A heading, about 8 ft. high and 12 ft. wide, was driven on the center line, with its bottom as nearly as possible on the grade line of the bottom of the tunnel. It was drilled in the ordinary manner by four drills mounted on two columns. The face of the headings varied from 10 to 30 ft. in advance of the cutting edge. After driving the heading for about 10 ft., the bottom was cleared out and a concrete cradle was set. The width of the cradles varied, but was generally from 8 to 10 ft.

The excavation was enlarged to full size as the shield advanced, the drills being mounted in the forward compartments of the shield, as shown by Fig. 1, Plate LXXXVII, which represents the conditions after the opening had been cut in the bulkhead, but before the new methods, mentioned later, had been developed.

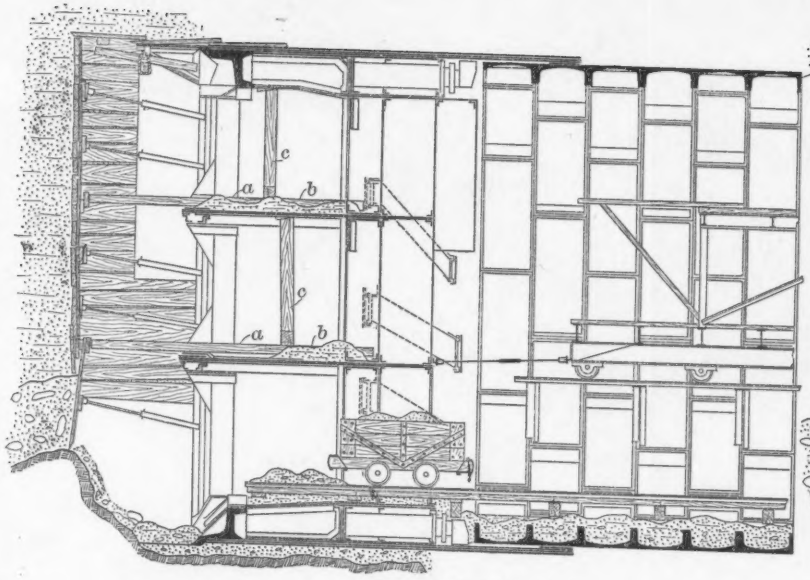
The sides and top were shot downward into the heading. The area of the face remaining behind the heading was large, and a great number of holes and several rounds were required to fire the face to advantage. As soon as firing was started at the face, the heading was completely blocked, and operations there had to be suspended until the mucking was nearly completed. The bottom-heading method was probably as good as any that could be devised for use with the shields as originally installed. All the muck had to be taken from the face by hand and handled through the chutes or doors. By drilling from the shield, some muck was blasted on to the extensions of the floors and could be handled from the upper compartments. At best, however, the shield with the closed transverse bulkhead was a serious obstacle to rapid work in rock sections.

The full-face method was only used where the rock was not considered safe for a heading. A cut was fired at the bottom, together with side holes, in a manner quite similar to that adopted in the first set of holes for a bottom heading. The cradle was then placed, in lengths of either 2.5 or 5 ft., after which the remainder of the face was fired in the same manner as for the bottom-heading method. The closed transverse bulkhead with air-locks, as shown in Fig. 1, Plate LXXXVI, was placed in the shield in the hope that it would only be necessary to maintain the full air pressure in the working compart-

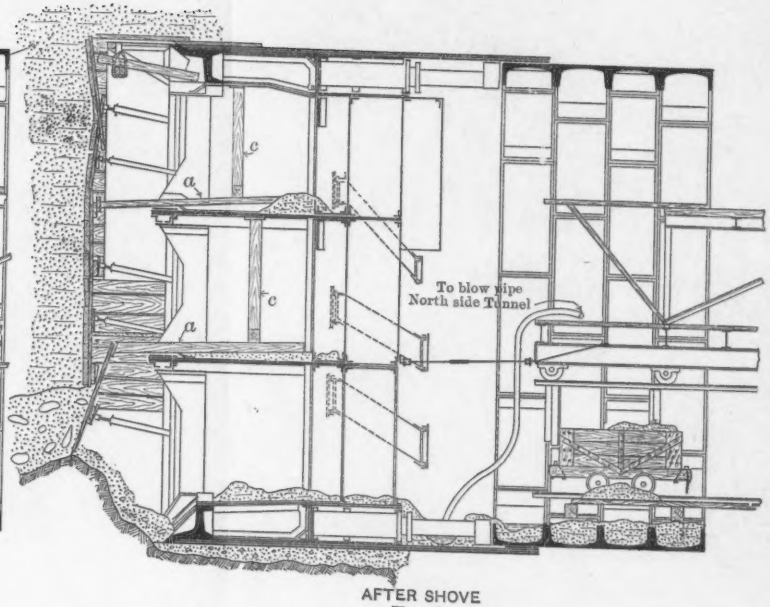
ments in front of the bulkhead. It was also thought that some form of bulkhead which could be closed quickly and tightly would be necessary to prevent flooding the tunnel in case of blows. While no attempt was ever made to reduce the pressure behind the shield bulkhead, it was obvious from the experience with Tunnels *B* and *D*, while working in the sand between Manhattan and the reef, that the plan was not practicable, and that the closed bulkhead in the bottom was a hindrance instead of a safeguard. As soon as rock was encountered in those tunnels at the west edge of the reef, the contractor cut through the bulkheads and altered them, as shown in Fig. 2, Plate LXXXVI.

Taking advantage of the experience gained, openings were cut through the bulkheads in Shields *A* and *C*, while they were shut down near the edge of the Manhattan ledge. In erecting the shields at Long Island City in May and June, 1906, openings were also provided. These shields had to pass through about 700 ft. of rock at the start, the greater portion of which was all-rock section. It was at that point that openings were first used extensively and methods were developed, which would not have been possible except where cars could be passed through the shield. The bottom-heading method was first tried, but the working space in front of the shield was cramped, and but few men could be employed in loading the cars. To give more room, the heading was gradually widened. The enlargement at the top, when made from the shield, blocked all work at the face of the heading while the former operation was in progress. To reduce the delays, the heading was raised, thus reducing the quantity of rock left in the top, and the bottom was taken out as a bench. To avoid blocking the tracks when firing the top, a heavy timber platform was built out from the floors of the middle working compartments. Most of the muck from the top was caught on the platform and dropped into cars below. This method of working is shown by Fig. 2, Plate LXXXVII. The platforms were not entirely satisfactory, and, later, the drills in the heading were turned upward and a top bench was also drilled and fired, as shown by Fig. 3, Plate LXXXVII. There was then so little excavation left in the top that the muck was allowed to fall on the tracks and was quickly cleared away. The method just outlined is called the center-heading method, and was the most satisfactory plan devised for full-rock sections.

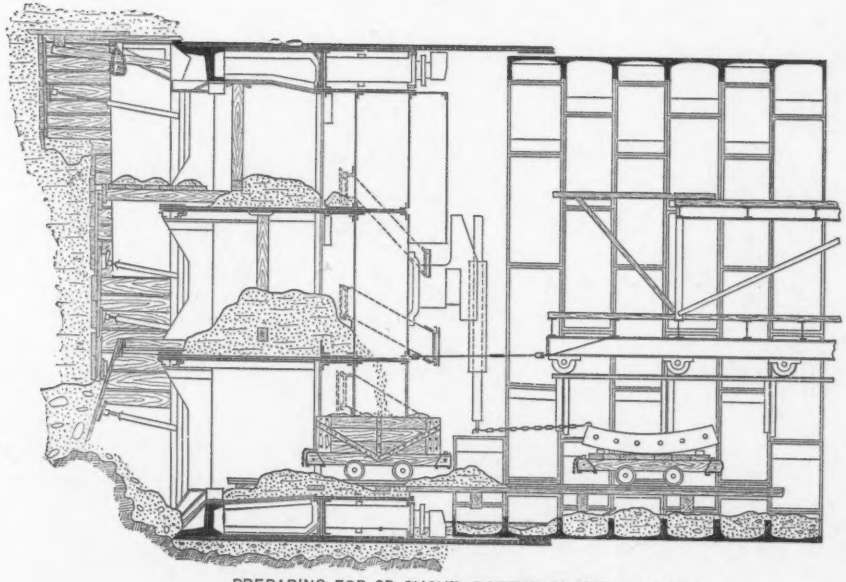
Excavation in Part Rock and Part Earth.—This was probably the



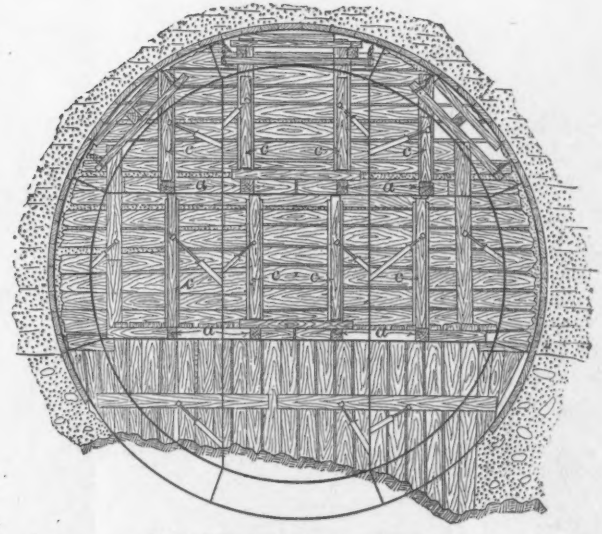
READY FOR SHOVE
FIG. 1.



AFTER SHOVE
FIG. 2.



PREPARING FOR 2D SHOVE. BOTTOM PLATES ERECTED.
FIG. 3.



ELEVATION SHOWING BREASTING
FIG. 4.

most difficult work encountered, particularly when the rock was covered with boulders and coarse sharp sand which permitted a free escape of air. It was necessary, before removing the rock immediately under the soft ground, to excavate the earth in advance of the shield to a point beyond where the rock was to be disturbed, and to support, in some way, the roof, sides, and face of the opening thus made. The hoods were designed mainly for the purpose of supporting the roof and the sides. With the fixed hood it was necessary either to excavate for the distance of the desired shove in front of it or else to force the hood into the undisturbed material. To avoid this difficulty, the sliding hoods were tried as an experiment.

In using the sliding hood, which will be described in detail in Mr. Japp's paper, the segments commencing at the top were forced forward by the screw rod, one at a time, as far as possible into the undisturbed material. Just enough material was then removed from underneath and in front of the section to free it, and it was again forced forward. These operations were repeated until the section had been extended far enough for a shove. As soon as two or three sections had been pushed forward in this way, the face near the advance end of the sliding hood was protected by a breast board set on edge and braced from the face. Gradually, all the segments were worked forward, and, at the same time, the whole soft ground face was sheeted with timber. At times polings were placed over the extended segments in order to make room for a second shove, as shown on Plate LXXXVIII. When the shield was advanced the nuts on the screw rods were loosened and the sections of the hoods were telescoped on to the shield. The idea was ingenious, but proved impracticable, because of the unequal relative movements of the top and bottom of the shield in shoving, bringing transverse strains on the hood sections.

With the fixed hood, poling boards were used to support the roof and sides, and the face was supported in the manner described for the sliding hoods. The polings were usually maple or oak planks, 2 in. thick, about 8 in. wide, and 6½ ft. long. In advancing the face, the top board of the old breast was first removed, then the material was carefully worked out for the length of the poling. The latter was then placed, with the rear end resting over the hood and the forward end forced as far as possible into the undisturbed material. When two or three polings had been placed, a breast board was set. After several

polings were in position, their forward ends were supported by some form of cantilever attached to the hood. Plate LXXXIX shows one kind of supports. In this way all the soft material was excavated down to the rock surface, and the roof, sides, and face were sheeted with timber. In shoving, the polings in the roof and sides were lost. It was found that the breast could usually be advanced 5 ft. with safety. The fixed hood made it possible to set the face about 7 or 8 ft. in front of the cutting edge without increasing the length of the polings. This distance was ample for two shoves, and was generally adopted, although a great many faces were set for one shove only.

Fixed hoods were substituted for those of the sliding type, originally placed on Shields *B* and *D* at Manhattan, at about the time the latter encountered the rock at the reef.

In placing the polings and breasting, all voids behind them were filled as far as possible with marsh hay or bags of sawdust or clay. To prevent loss of air in open material, the joints between the boards were plastered with clay especially prepared for the purpose in a pug mill. The sliding extensions to the floors of the working compartments were often used, in the early part of the work, to support the timber face or loose rock, as shown in Fig. 1, Plate LXXXVIII. At such times the front of the extensions was held tightly against the planking by the pressure of the floor jacks. While shoving, the pressure on the floor jacks was gradually released, allowing the floors to slide back into the shield and still afford support to the face. The extensions also afforded convenient working platforms. They were subject to severe bending strains while the shield was being shoved, however, and the cast-iron rams were frequently broken or jammed. The extensions did not last beyond the edge of the ledge at Manhattan, nor more than about half through the rock work at Long Island City. The fixed extensions originally placed on Shields *A* and *C* at Manhattan were not substantial enough, and lasted only a few days.

Wherever the rock face was sufficiently sound and high, a bottom heading was driven some 20 or 30 ft. in advance of the shield. The heading was driven and the cradle placed independently of the face of the soft ground above, and in the manner described for all-rock sections. The remainder of the rock face was removed by firing top and side rounds into the bottom heading after the soft ground had been excavated. Great care had to be taken in firing in order not to disturb

FIG. 1.—SMALL SHAFT SUNK TO ROCK.

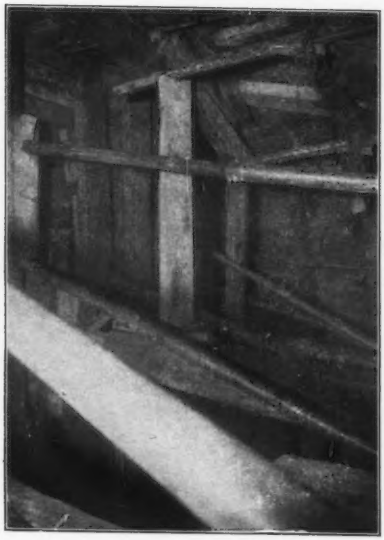


FIG. 2.—BREASTING AND POLING IN FRONT OF SHIELD.

FIG. 3.—SHUTTERS ON FRONT OF SHIELD.

FIG. 4.—HYDRAULIC ERECTOR PLACING SEGMENT.

the timber work or break the rock away from under the breast boards. If either occurred, a serious run was likely to follow. The bottom-heading method is shown by Figs. 1, 2, and 3, Plate LXXXVIII, and the breasting and poling by Fig. 2, Plate XC.

In the early part of the work, where a bottom heading was impracticable, the soft ground was first excavated as described above, and the rock was drilled by machines mounted on tripods, and fired as a bench. By this plan no drilling could be done until the soft ground was removed. This is called the rock-bench method.

Later the rock-cut method was devised. Drills were set up on columns in the bottom compartments of the shield, and the face was drilled while work was in progress on the soft ground above. The drilling was done either for a horizontal or vertical cut and side and top rounds. The drillers were protected while at work by platforms of timber built out from the floors of the compartments above. This plan, while probably not quite as economical of explosives, saved nearly all the delay due to drilling the bench.

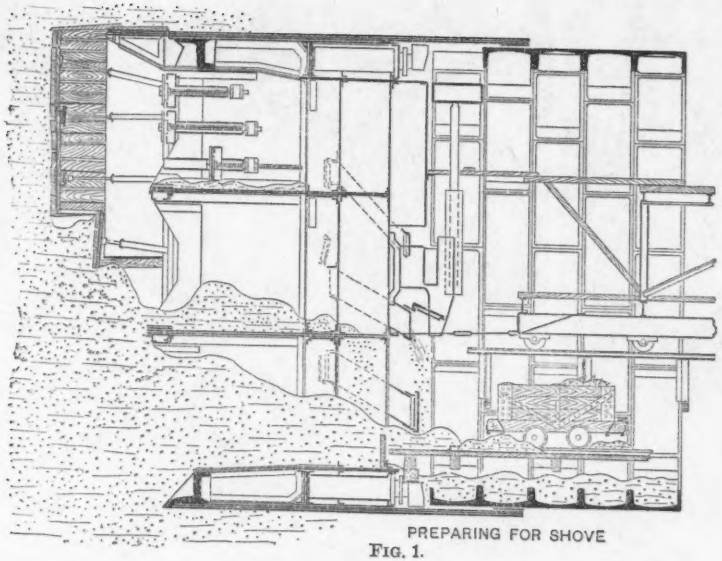
All-Earth Section.—As described by Messrs. Hay and Fitzmaurice, in a paper on the Blackwall Tunnel,* the contractor had used, with marked success, shutters in the face of the shield for excavating in loose open material. He naturally adopted the method for the East River work. When the shields in Tunnels *B* and *D*, at Manhattan, the first to be driven through soft ground, reached a point under the actual bulkhead line, work was partly suspended and shutters were put in place in the face of the top and center compartments. The shutters in the center compartments in Shield *D* are shown in Fig. 3, Plate XC, while the method of work with the shutters is shown by Figs. 4, 5, 6, and 7, Plate LXXXVIII. Fig. 4 on that plate shows the shield ready for a shove. As the pressure was applied to the shield jacks, men loosened the nuts on the screws holding the ends of the shutters, and allowed the latter to slide back into the working compartments. At the end of the shove, the shutters were in the position shown in Fig. 5, Plate LXXXVIII. In preparing for a new shove, the slides in the shutters were opened, and the material in front was raked into the shield. At the same time, the shutters were gradually worked forward. The two upper shutters in a compartment were generally advanced from 12 to 15 in., after which the muck could be

* *Minutes of Proceedings*, Inst. C. E., Vol. CXXX, p. 50.

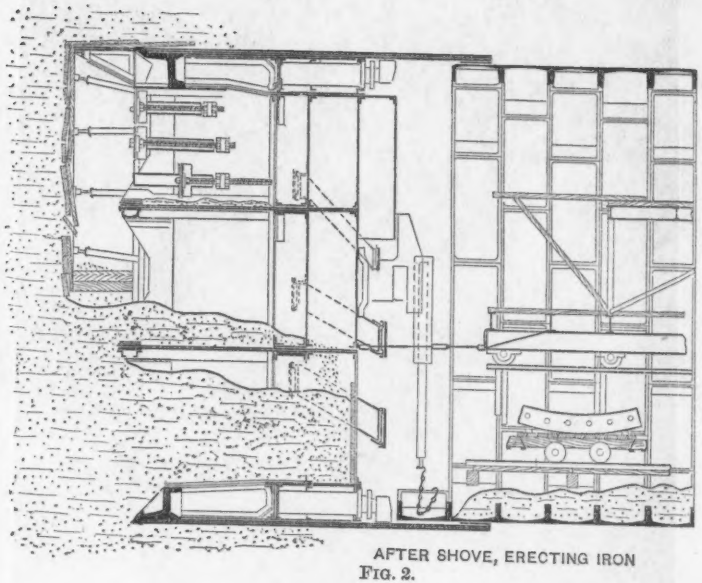
shoveled out over the bottom shutters, as shown on Fig. 6, Plate LXXXVIII, and Fig. 3, Plate XC. No shutters were placed in the bottom compartments, and as the air pressure was not generally high enough to keep the face dry at the bottom, these compartments were pretty well filled with the soft, wet quicksand. Just before shoving, this material was excavated to a point where it ran in faster than it could be taken out. Much of the excavation in the bottom compartment was done by the blow-pipe. During the shove the material from the bottom compartment often ran back through the open door in the transverse bulkhead, as shown by Fig. 5, Plate LXXXVIII.

In the Blackwall Tunnel the material was reported to have been loose enough to keep in close contact with the shutters at all times. In the East River Tunnels this was not the case. The sand at the top was dry and would often stand with a vertical face for some hours. In advancing the shutters, it was difficult to bring them into close contact with the face at the end of the operation. The soft material at the bottom was constantly running into the lower compartment and undermining the stiff dry material at the top. The latter gradually broke away, and, at times, the actual face was some feet in advance of the shutters. Under those circumstances, the air escaped freely through the unprotected sand face. The joints of the shutters were plastered with clay, but this did not keep the air from passing out through the lower compartments. This condition facilitated the formation of blows, which were of constant occurrence where shutters were used in the sand. In Tunnels *B* and *D*, at Manhattan, the shutters were used in the above manner clear across to the reef. In Tunnel *C*, which was considerably behind Tunnels *B* and *D*, the shutters, although placed, were never used against the face, and the excavation was carried on by poling the top and breasting the face. The change resulted in much better progress and fewer blows. The excavation through the soft material in Tunnel *C* had just been completed when Tunnel *A* was started, and the gangs of workmen were exchanged.

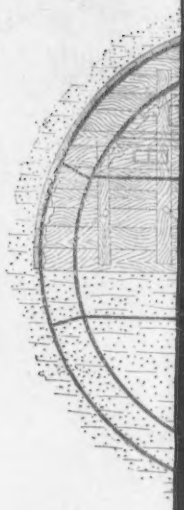
The work in soft ground in Tunnel *A* thus gained the benefit of the experience in Tunnel *C*. Shutters were placed only in the top compartments in this tunnel, and, as in Tunnel *C*, were never used in contact with the face. The method of work is shown by Figs. 1, 2, and 3, Plate XCI. The result was still more rapid progress in Tunnel *A*, and although the loss of air was fully as great in this tunnel as in



PREPARING FOR SHOVE
FIG. 1.



AFTER SHOVE, ERECTING IRON
FIG. 2.



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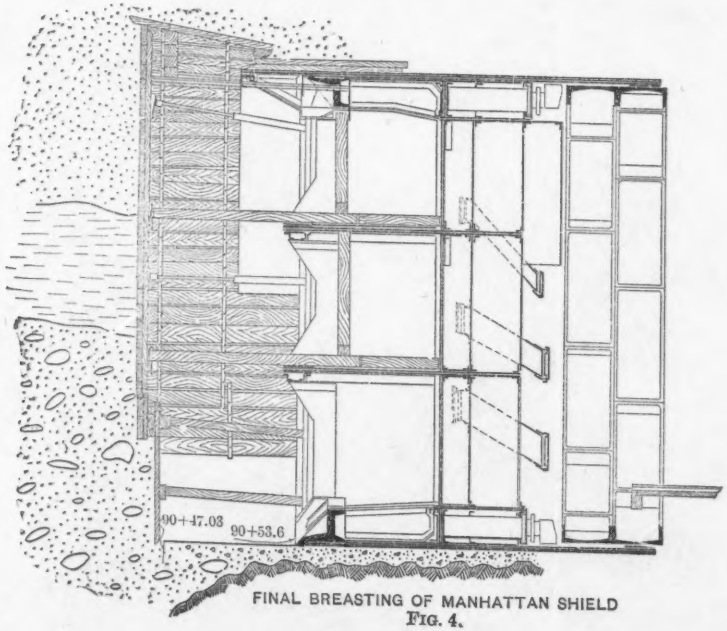
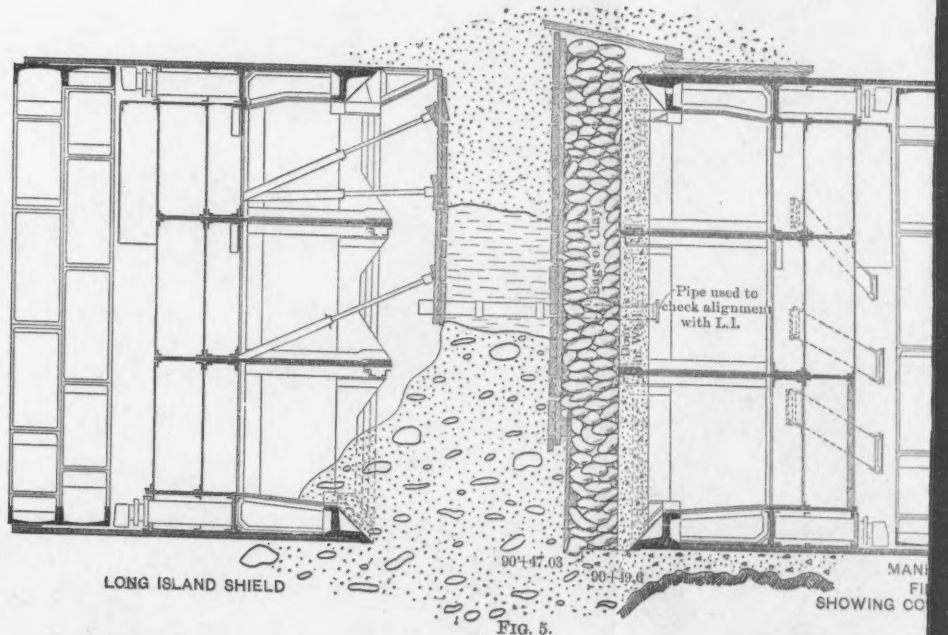


FINAL BREASTING OF MANHATTAN SHIELD
FIG. 4.



LONG ISLAND SHIELD
FIG. 5.
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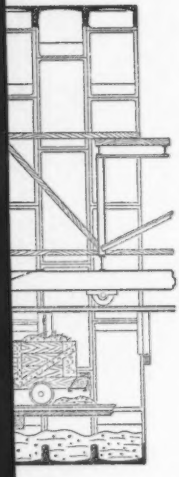
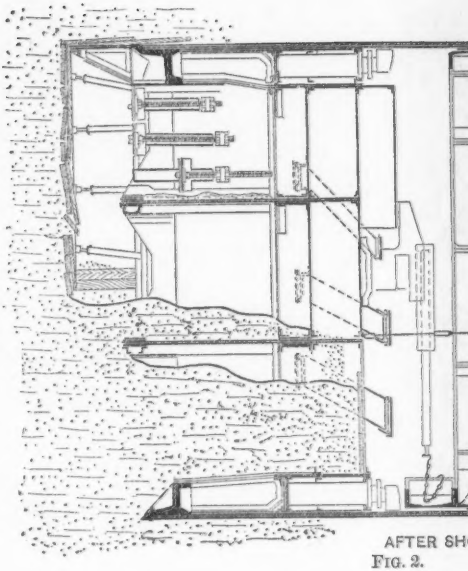
SHOWING CO

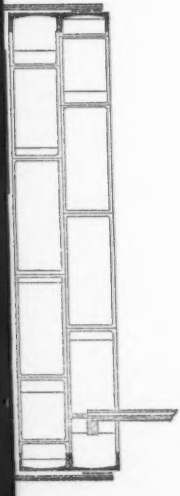


SHOVE

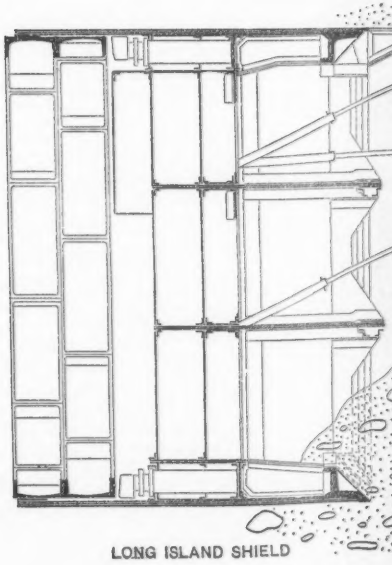


AFTER SH

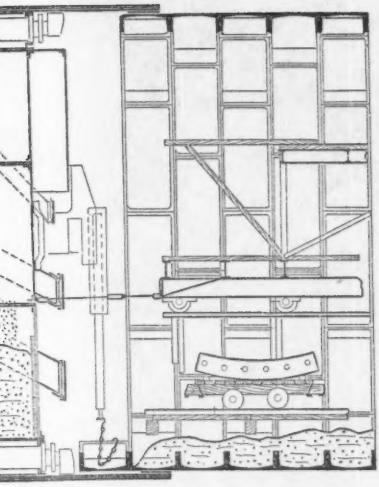
FIG. 2.



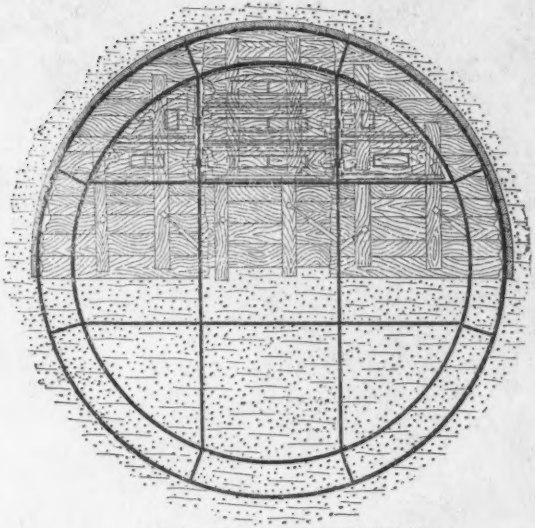
SHIELD



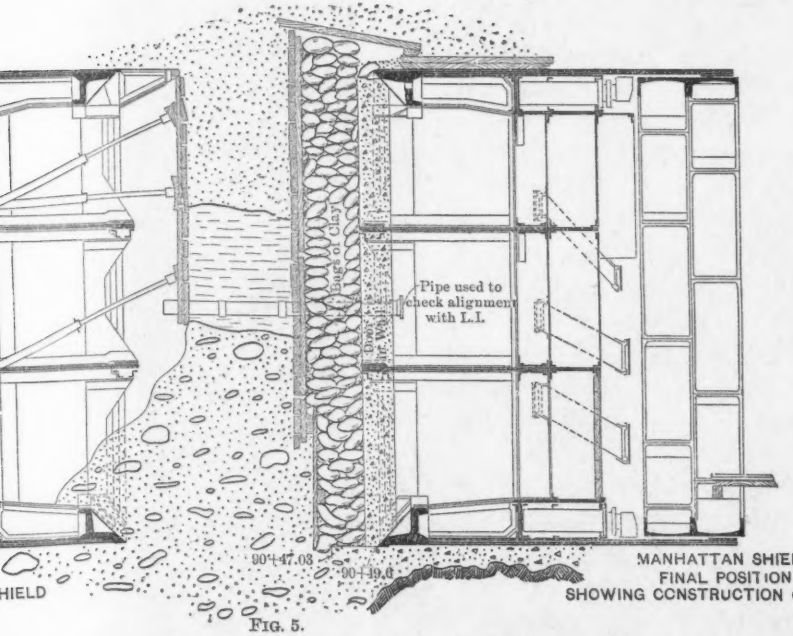
LONG ISLAND SHIELD



AFTER SHOVE, ERECTING IRON
FIG. 2.



SECTION:- SHUTTERS AND BREASTING
FIG. 3.



HIELD

MANHATTAN SHIELD
FINAL POSITION
SHOWING CONSTRUCTION OF BULKHEAD

FIG. 5.

the other three, there was only one blow which caused any considerable loss of pressure. In Tunnels *A* and *C* the diaphragms in the rear of the center compartments of the lower tiers of working chambers were removed before the shields entered the soft ground. The change was not of as much advantage in soft ground as in rock, but it facilitated the removal of the soft wet sand in the bottom. In Tunnel *A*, after encountering gravel, a belt conveyor was suspended from the traveling stage with one end projecting through the opening into the working compartment. The use of the conveyor made it possible to continue mucking at the face while the bottom plates of the iron lining were being put in place, and resulted in a material increase in the rate of progress.

The shutters were not placed on the Long Island shields at all. Just before the shields passed into all soft ground, a fixed hood was attached to each.

The method of working in soft ground from Long Island City is illustrated by Plate XCII. The full lines at the face of the shield show the position of the earth before a shove of the shield, and the dotted lines show the same after the shove. The face was mined out to the front of the hood and breasted down to a little below the floor of the top pockets of the shield. In the middle pocket the earth was allowed to take its natural slope back on the floor. Toward the rear of the bottom pockets it was held by stop-planks. The air pressure was always about equal to the hydrostatic head at the middle of the shield, so that the face in the upper and middle pockets was dry. In the lower pockets it was wet, and flowed under the pressure of shoving the shield. By this method 4 195 lin. ft. of tunnel was excavated by the four Long Island shields in 120 days, from November 1st, 1907, to March 1st, 1908. This was an average of 8.74 ft. per day per shield.

The rate of progress, the nature of the materials, and the methods adopted are shown in Table 2.

Preparations for Junction of Shields.—As previously mentioned, the Manhattan shields were stopped at the edge of the reef. Before making the final shove of those shields, special polings were placed with unusual care. The excavation was bell-shaped to receive the Long Island shields. The arrangement of the polings is shown by Figs. 4 and 5, Plate XCI. After the shields were shoved into final position, as shown at the right in Fig. 5, the rear end of the polings

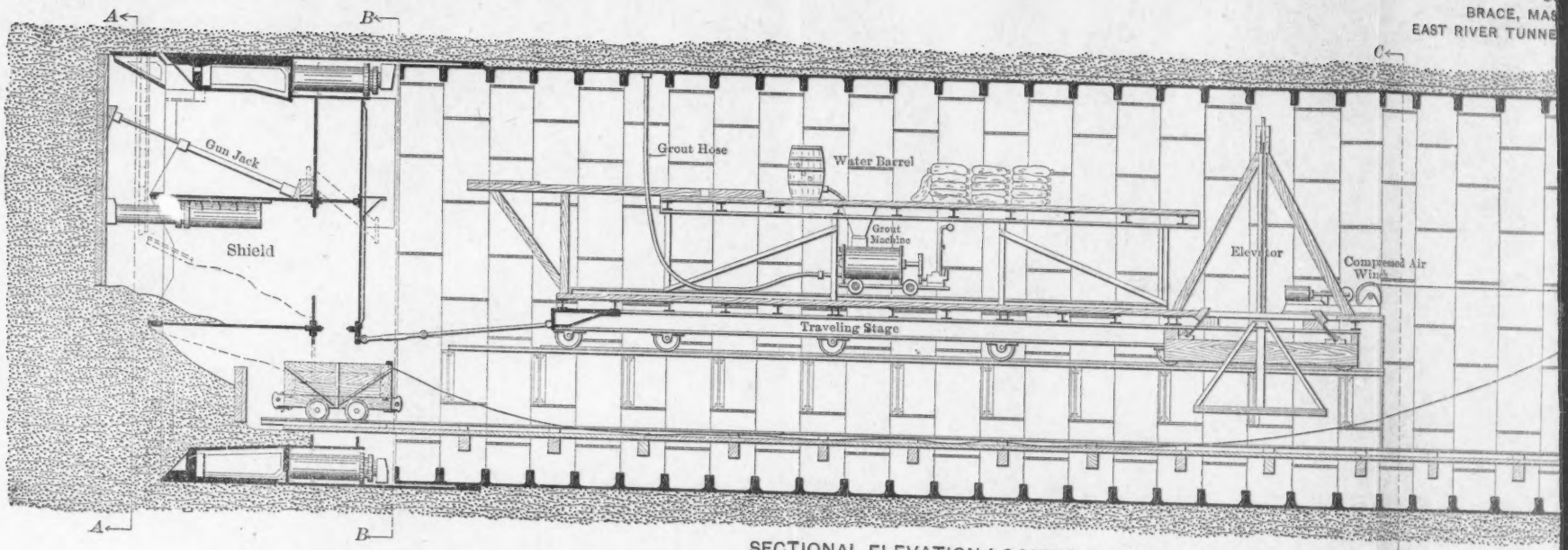
TABLE 2.—(*Continued.*)

LINE D, LONG ISLAND.

Material.	Method.	Station: From	Station: To	Date: From	Date: To	Number of days.	Linear feet.	Rate of progress, in feet per day.	Remarks.
Rock	Bottom heading	68 + 50.6	69 + 77	June 2, '06	Oct. 24, '06	144	126.4	0.87	
Earth and rock.	Bottom heading	69 + 77	71 + 22	Oct. 24, '06	Jan. 13, '07	81	145	1.79	
All rock	Bottom heading	71 + 22	72 + 00	Jan. 13, '07	Mar. 3, '07	49	78	1.59	
All rock	Center heading	72 + 00	75 + 73	Mar. 3, '07	July 10, '07	129	373	2.89	
Earth and rock.	Going out of rock	75 + 73	77 + 63	July 10, '07	Sept. 25, '07	77	190	2.47	
All earth	Soft ground	77 + 63	90 + 38.6	Sept. 25, '07	Mar. 7, '08	164	1275.6	7.78	

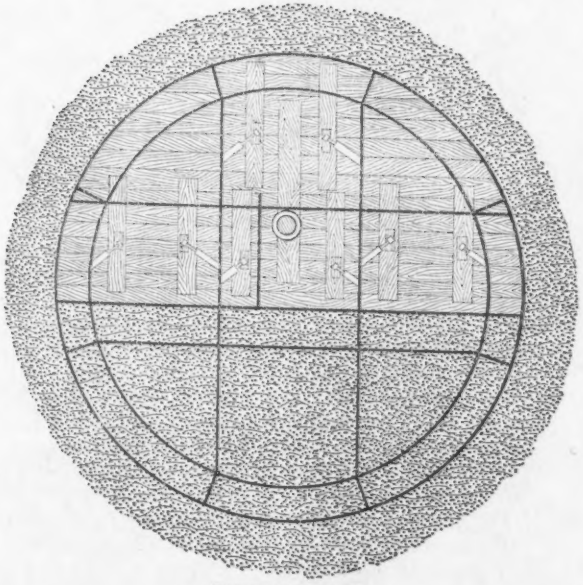
LINE A, MANHATTAN.

Material.	Method.	Station: From	Station: To	Date: From	Date: To	Number of days.	Linear feet.	Rate of progress, in feet per day.	Remarks.
Rock	Top heading	108 + 43	107 + 74	July 20, '05	Aug. 3, '05	14	69	1.21	Excavation in normal air, and before advance of shield.
	Top lift of bench	108 + 43	107 + 74	Aug. 8, '05	Aug. 23, '05	15 (57)			
	Bottom lift of bench	108 + 43	107 + 74	Aug. 30, '05	Sept. 27, '05	28			
Rock	Bottom heading	107 + 74	107 + 21	Sept. 27, '05	Oct. 23, '05	26	53	0.96	Bottom heading timbered to avoid the possibility of a break.
	Bottom heading	107 + 74	107 + 21	Nov. 30, '05	Dec. 29, '05	29 (55)			
Mixed	Bottom heading	107 + 21	106 + 99	Oct. 26, '06	Nov. 20, '06	25	22	0.88	Bottom heading timbered.
Mixed	Rock bench	106 + 99	106 + 34	Nov. 20, '06	Jan. 13, '07	54	65	1.20	
Earth	Poling and breasting	106 + 34	99 + 11	Jan. 13, '07	Apr. 17, '07	94	723	7.69	
Mixed	Rock cut	99 + 11	93 + 96	Apr. 17, '07	Oct. 24, '07	190	515	2.71	
Rock	Bottom heading	93 + 96	93 + 58	Oct. 24, '07	Nov. 14, '07	21	38	1.81	
Rock	Center heading	93 + 58	92 + 42	Nov. 14, '07	Dec. 27, '07	46	116	2.52	
Rock	Bottom heading	92 + 42	91 + 05	Dec. 27, '07	Feb. 24, '08	59	137	2.32	
Mixed	Rock cut	91 + 05	90 + 57	Feb. 24, '08	Mar. 20, '08	25	48	1.92	



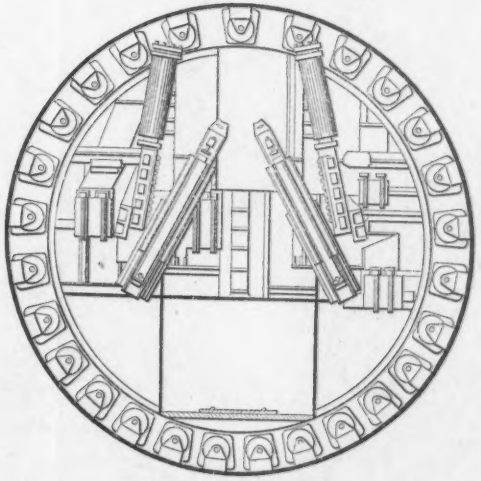
SECTIONAL ELEVATION LOOKING NORTH

FIG.1



SECTION *A-A*

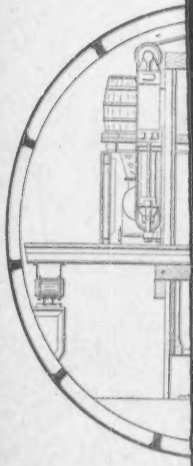
FIG.2



SECTION *B-B*

SHOWING BACK VIEW OF SHIELD

FIG.3



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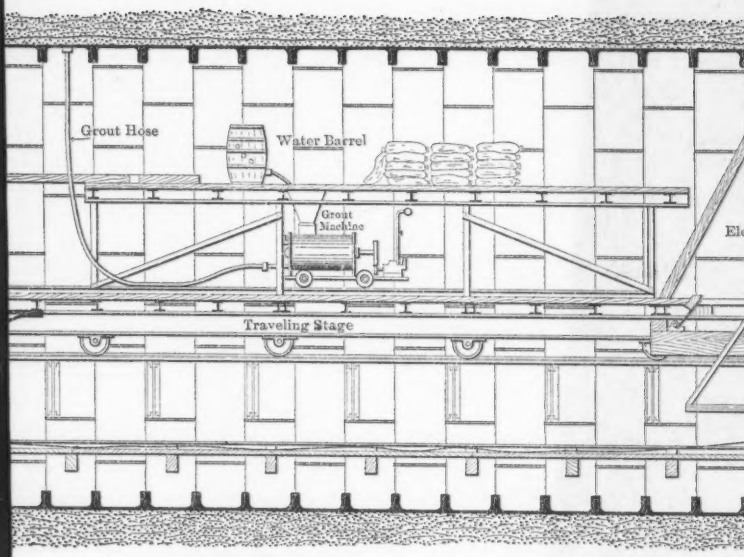
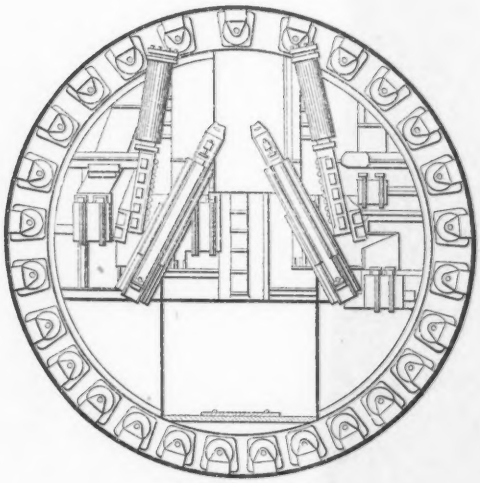


SECTIONAL ELEVATION LOOKING NORTH

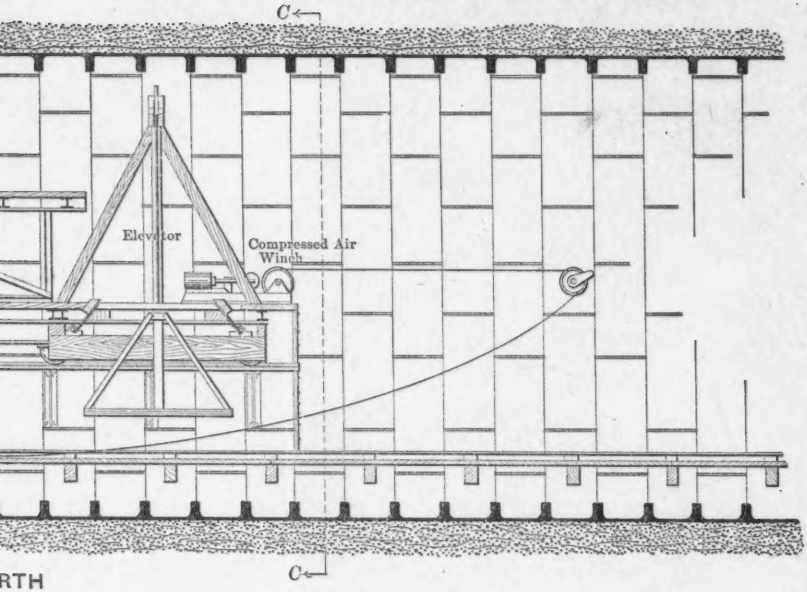
FIG.1



SECTION *B-B*

SHOWING BACK VIEW OF SHIELD

FIG.3



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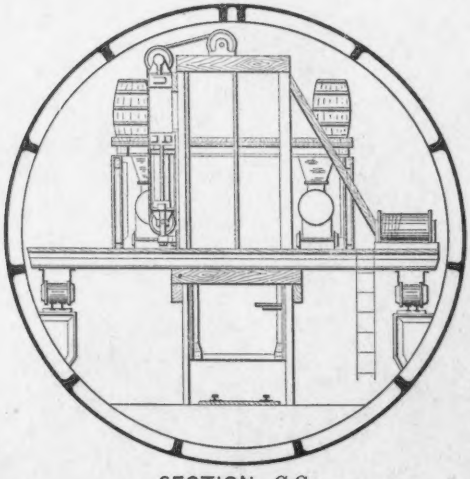

SECTION *C-C*
SHOWING BACK VIEW OF TRAVELING STAGE
FIG. 4

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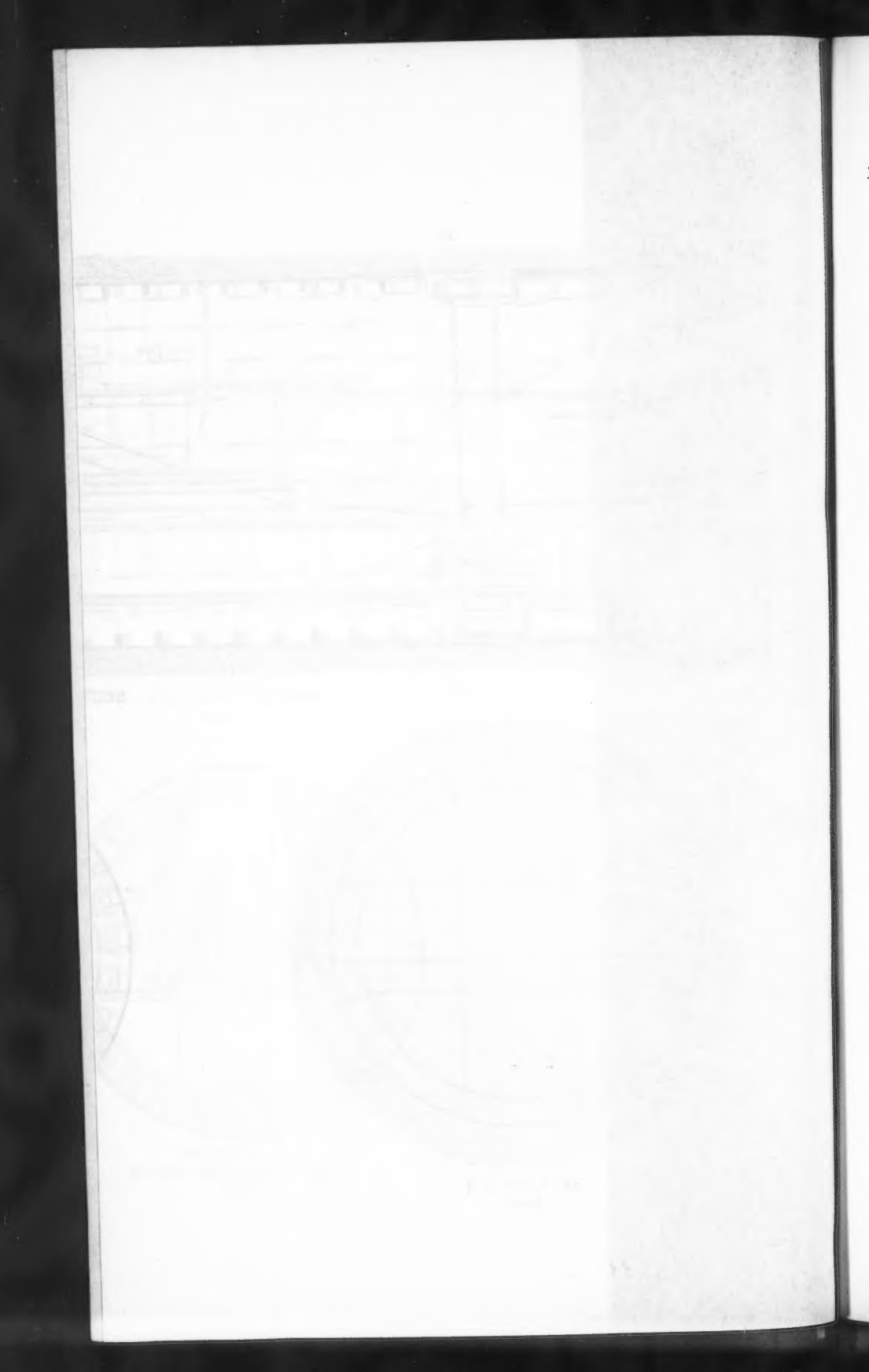


TABLE 2.—Rate of Progress, Nature of Materials, and Methods Adopted in Construction of East River Tunnels.

Line A, Long Island.

Material.	Method.	Station: From	Station: To	Date: From	Date: To	Number of days.	Linear feet.	Rate of progress, in feet per day.	Remarks.
All rock.....	Bottom heading	69 + 39.9	69 + 79	Aug. 2, '06	Sept. 25, '06	54	39.1	0.724	
All rock.....	Center heading.....	69 + 79	70 + 64	Sept. 25, '06	Nov. 21, '06	57	85	1.49	
Earth and rock.	Center heading.....	70 + 64	71 + 34	Nov. 21, '06	Dec. 30, '06	39	70	1.79	
Earth and rock.	Bottom heading.....	71 + 34	71 + 89	Dec. 30, '06	Feb. 13, '07	45	55	1.22	
All rock.....	Bottom heading.....	71 + 89	72 + 11	Feb. 13, '07	Feb. 21, '07	8	22	2.75	
Earth and rock.	Center heading.....	72 + 11	72 + 67	Feb. 21, '07	Mar. 19, '07	26	56	2.15	
All rock.....	Center heading.....	72 + 67	76 + 54	Mar. 19, '07	Sept. 6, '07	171	387	2.26	
Earth and rock.	Going out of rock...	76 + 54	77 + 24	Sept. 6, '07	Oct. 4, '07	28	70	2.50	
All earth	Soft ground.....	77 + 24	90 + 57.3	Oct. 4, '07	Mar. 26, '08	174	1 333.3	7.66	

Line B, Long Island.

Material.	Method.	Station: From	Station: To	Date: From	Date: To	Number of days.	Linear feet.	Rate of progress, in feet per day.	Remarks.
All rock.....	Bottom heading	69 + 29.6	70 + 46	Oct. 16, '06	Nov. 20, '06	35	116.4	3.33	
Earth and rock.	Bottom heading.....	70 + 46	71 + 95	Nov. 20, '06	Feb. 23, '07	95	149	1.57	
All rock.....	Bottom heading.....	71 + 95	72 + 25	Feb. 23, '07	Mar. 6, '07	11	30	2.73	
Earth and rock.	Center heading.....	72 + 25	72 + 60	Mar. 6, '07	Mar. 24, '07	18	35	1.94	
All rock.....	Going out of rock....	72 + 60	76 + 57	Mar. 24, '07	Aug. 7, '07	136	397	2.92	
Earth and rock.	Soft ground.....	76 + 57	77 + 30	Aug. 7, '07	Sept. 5, '07	29	73	2.52	
All earth	Soft ground.....	77 + 30	90 + 49.6	Sept. 5, '07	Mar. 19, '08	196	1 319.6	6.73	

Line C, Long Island.

Material.	Method.	Station: From	Station: To	Date: From	Date: To	Number of days.	Linear feet.	Rate of progress, in feet per day.	Remarks.
All rock.....	Bottom heading.....	68 + 61.9	69 + 93	June 11, '06	Oct. 16, '06	127	131.1	1.03	
Earth and rock.	Bottom heading.....	69 + 93	71 + 65	Oct. 16, '06	Feb. 7, '07	114	172	1.51	
All rock.....	Bottom heading.....	71 + 65	71 + 91	Feb. 7, '07	Feb. 13, '07	6	26	4.33	
All rock.....	Center heading.....	71 + 91	75 + 81	Feb. 13, '07	July 20, '07	157	390	2.48	
Earth and rock.	Going out of rock...	75 + 81	76 + 56	July 20, '07	Aug. 25, '07	36	75	2.08	
All earth	Soft ground.....	76 + 56	90 + 44.4	Aug. 25, '07	Mar. 17, '08	205	1 388.4	6.77	

TABLE 2.—(*Continued.*)

LINE C, MANHATTAN.—(*Continued.*)

Material.	Method.	Station: From	Station: To	Date: From	Date: To	No. of days.	Linear feet.	Rate of progress, in feet per day.	Remarks.
Mixed	Rock cut	106 + 17	105 + 85	Apr. 2, '06	Apr. 20, '06	18	32	1.78	Shut down on account of air shortage.
Mixed	Rock cut	105 + 85	105 + 55	July 27, '06	Aug. 26, '06	30	30	1.00	Shut down April 20th to July 27th, 1906.
Earth	Breasting and poling	105 + 55	99 + 40	Aug. 26, '06	Jan. 2, '07	127	615	4.84	
Mixed	Rock cut	99 + 40	98 + 70	Jan. 2, '07	Feb. 6, '07	35	70	2.00	
Rock	Full face	98 + 70	98 + 60	Feb. 6, '07	Feb. 12, '07	6	10	1.66	
Mixed	Bottom heading	98 + 60	98 + 39	Feb. 12, '07	Mar. 6, '07	22	21	0.95	
Rock	Bottom heading	98 + 39	98 + 17	Mar. 6, '07	Mar. 15, '07	9	22	2.44	
Mixed	Rock cut	98 + 17	95 + 68	Mar. 15, '07	July 30, '07	110	249	2.26	Heading advanced to 97 + 82.
Rock	Middle heading	95 + 68	94 + 61	July 30, '07	Aug. 21, '07	49	107	2.18	" " " 94 + 35.
Mixed	Rock cut	94 + 61	93 + 56	Aug. 21, '07	Oct. 3, '07	43	106	2.46	
Rock	Middle heading	93 + 56	92 + 73	Oct. 3, '07	Nov. 11, '07	39	83	2.13	
Mixed	Rock cut	92 + 73	90 + 55	Nov. 11, '07	Feb. 13, '08	94	218	2.32	Shut down until Line D shields met.
Mixed	Rock cut	90 + 55	90 + 44.4	Feb. 25, '08	Mar. 3, '08	6	11	1.83	

LINE D, MANHATTAN.

Material.	Method.	Station: From	Station: To	Date: From	Date: To	No. of days.	Linear feet.	Rate of progress, in feet per day.	Remarks.
Rock	Top heading	107 + 70.49	107 + 16	Dec. 9, '04	Jan. 31, '05	123	90	0.73	In normal air.
	Removing bench	107 + 70.49	107 + 35	Jan. 1, '05	Jan. 27, '05				
	Bottom heading	107 + 35	106 + 80	Jan. 30, '05	Feb. 10, '05				
	Trimming	107 + 70	106 + 80	Mar. 29, '05	Apr. 12, '05				
	Trimming	107 + 70	106 + 80	Aug. 31, '05	Sept. 19, '05				
Rock	Bottom heading	106 + 80	106 + 67	Oct. 5, '05	Nov. 8, '05	34	13	0.40	Bottom heading timbered.
Mixed	Bottom heading	106 + 67	106 + 29	Nov. 8, '05	Dec. 23, '05	45	38	0.84	
Mixed	Sliding hood and breasting. Rock bench	106 + 29	105 + 70	Dec. 23, '05	Jan. 24, '06	32	59	1.84	
Earth	Poling and breasting	105 + 70	104 + 61	Jan. 24, '06	Feb. 27, '06	31	109	3.41	
Earth	Poling, breasting and shutters	104 + 61	103 + 90	Mar. 2, '06	Mar. 31, '06	29	71	2.45	Three days' delay to set shutters in top. Shut down 20 days to permit consolidation of the river bed and to repair broken plates.

TABLE 2.—(*Continued.*)

LINE B, MANHATTAN.

Material.	Method.	Station: From	Station: To	Date: From	Date: To	Number of days.	Linear feet.	Rate of progress, in feet per day.	Remarks.
Rock	Top heading	108 + 35	107 + 87	July 6, '05	July 27, '05	21			
	Top lift of bench	108 + 35	107 + 87	Aug. 3, '05	Aug. 14, '05	11			
	Bottom lift of bench	108 + 35	108 + 15	Aug. 26, '05	Aug. 30, '05	4	51	48	0.94	Excavation done in normal air and before advance of shield.
	Bottom lift of bench	108 + 15	107 + 87	Sept. 11, '05	Sept. 26, '05	15			
Rock	Bottom heading	107 + 87	107 + 00	Oct. 23, '05	Jan. 17, '06	86		87	1.01	
Mixed	Bottom heading	107 + 00	106 + 64	Jan. 17, '06	Feb. 12, '06	26		36	1.38	
Mixed	Rock bench	106 + 64	106 + 31	Feb. 12, '06	Mar. 1, '06	17		33	1.94	
Earth	Poling and breasting	106 + 31	105 + 58	Mar. 1, '06	Apr. 3, '06	33		73	2.21	
Earth	Shutters in contact with face	105 + 58	99 + 19	Apr. 9, '06	Nov. 1, '06	206		639	3.10	
Mixed	Rock bench	99 + 19	98 + 44	Nov. 1, '06	Dec. 29, '06	58		75	1.30	
Mixed	Bottom heading	98 + 44	97 + 76	Dec. 29, '06	Feb. 12, '07	45		68	1.51	
Mixed	Rock cut	97 + 66	93 + 84	Feb. 12, '07	Aug. 6, '07	175		392	2.24	
Rock	Full face	93 + 84	93 + 21	Aug. 6, '07	Sept. 2, '07	27		63	2.33	
Rock	Center heading	93 + 21	92 + 30	Sept. 2, '07	Oct. 12, '07	40		91	2.28	
Rock	Bottom heading	92 + 30	90 + 99	Oct. 12, '07	Dec. 6, '07	55		131	2.38	
Mixed	Rock cut	90 + 99	90 + 49.6	Dec. 6, '07	Jan. 3, '08	28		49.4	1.76	

LINE C, MANHATTAN.

Material.	Method.	Station: From	Station: To	Date: From	Date: To	Number of days.	Linear feet.	Rate of progress, in feet per day.	Remarks.
Rock	Top heading	107 + 79.03	107 + 69	Dec. 20, '04	Dec. 27, '04	7			
	Top heading	107 + 69	107 + 23	Jan. 1, '05	Jan. 15, '05	14			
	Excavating bench	107 + 79	107 + 23	Jan. 21, '05	Feb. 28, '05	38	54	0.77	Stopped to brace portal. No work done from March 12th to October 11th, 1905, except a little trimming in September. All work up to this date done in normal air. Heading advanced to 106 + 70 and bulkheaded.
	Bottom heading	107 + 23	106 + 72	Mar. 1, '05	Mar. 11, '05	10			
	Bottom heading	107 + 23	107 + 15	Oct. 12, '05	Oct. 27, '05	15			
Rock	Bottom heading	107 + 15	106 + 62	Nov. 6, '05	Dec. 2, '05	26	53	2.04	
Mixed	Bottom heading	106 + 62	106 + 55	Dec. 2, '05	Dec. 23, '05	21	7	0.33	Heading advanced to 106 + 40. Shut down in order that Line D might have a lead.
Mixed	Bottom heading	106 + 55	106 + 17	Feb. 12, '06	Mar. 22, '06	38	38	1.00	Shut down on account of air shortage.

TABLE 2.—(*Continued.*)

LINE C, MANHATTAN.—(*Continued.*)

TABLE 2.—(*Continued.*)

LINE D, MANHATTAN.—(*Continued.*)

Material.	Method.	STATION:		DATE:		Number of days.	Linear feet.	Rate of progress, in feet per day.	Remarks.
		From	To	From	To				
Earth.....	Shutters.....	103 + 90	99 + 41	Apr. 20, '06	Sept. 3, '06	136	449	3.40	Four days of 136, delay account of flood.
Mixed.....	Bottom bench.....	99 + 41	99 + 17	Sept. 3, '06	Sept. 23, '06	20	24	1.20	
Mixed.....	Bottom heading	99 + 17	98 + 50	Oct. 2, '06	Nov. 24, '06	53	67	1.27	Thirteen days' shut-down to put on hood.
Rock.....	Bottom heading.....	98 + 50	97 + 72	Nov. 24, '06	Jan. 16, '07	53	78	1.47	
Mixed.....	Bottom heading.....	97 + 72	97 + 27	Jan. 16, '07	Feb. 10, '07	25	45	1.40	
Mixed.....	Rock cut.....	97 + 27	95 + 72	Feb. 10, '07	Apr. 23, '07	72	155	2.15	
Rock.....	Middle heading.....	95 + 72	95 + 57	Apr. 23, '07	May 11, '07	18	15	0.83	
Rock	Middle heading.....	95 + 57	94 + 65	May 23, '07	June 17, '07	25	92	3.68	Twelve days' delay to repair cutting edge.
Mixed	Middle heading.....	94 + 65	94 + 41	June 17, '07	June 25, '07	8	24	3.00	
Mixed.....	Rock cut.....	94 + 41	94 + 03	June 25, '07	July 13, '07	18	38	2.11	
Rock.....	Middle heading.....	94 + 03	92 + 64	July 13, '07	Sept. 12, '07	61	139	2.28	
Mixed.....	Middle heading	92 + 64	92 + 54	Sept. 12, '07	Sept. 20, '07	8	10	1.25	
Rock.....	Middle heading.....	92 + 54	92 + 50	Sept. 20, '07	Sept. 21, '07	1	4	4.00	
Mixed.....	Rock cut.....	92 + 50	90 + 38.66	Sept. 21, '07	Jan. 8, '08	109	211.34	1.94	

rested over the cutting edge and allowed room for the removal of the hood. After the latter had been accomplished, the temporary bulkheads of concrete and clay bags were built as a precaution against blows when the shields were close together. An 8-in. pipe was then driven forward through the bulkhead for distances varying from 30 to 100 ft., in order to check the alignment and grade between the two workings before the shields were actually shoved together. The errors in the surveys were negligible, but here, as elsewhere, the shields were not exactly in the desired position, and it took careful handling to bring the cutting edges together. The Long Island shields were driven to meet those from Manhattan.

Openings were made between the headings as follows:

Tunnel *D*, February 20th, 1908;
Tunnel *B*, March 3d, 1908;
Tunnel *C*, March 5th, 1908;
Tunnel *A*, March 18th, 1908.

It was necessary to cut away the projecting floors of the working compartments before the cutting edges could be shoved together.

Contractor's Organization.—Tunnel operations were carried on continuously for thirteen days out of fourteen, regular work being shut down for repairs on alternate Sundays. When the required pressure was more than 32 lb., four gangs of laborers were employed, each gang working two shifts of 3 hours each, with an intermission of 3 hours between the shifts. When the pressure was less than 32 lb., three gangs were employed, each gang covering 8 hours, but with an intermission of about ½ hour in low pressure for lunch.

Air Pressures Required.—During the greater portion of the work in soft ground, pressure was maintained which would about balance the hydrostatic head at the axis of the tunnel. This required a pressure varying from 30 to 34 lb. per sq. in. above that of the atmosphere. In Tunnels *B* and *D*, at Manhattan, during the work in soft ground, pressures as high as 37 lb. were maintained for considerable periods of time; in the firm material near the reef 28 lb. was often sufficient. While removing the broken plates, the pressure was raised for a short time to 42 lb., and was maintained between 37½ and 40 lb. for a little more than one month.

Air Supply.—For regular operation the contractor furnished four

compressors on each side of the river, each having a rated capacity of 5 000 cu. ft. of free air per minute delivered at 50 lb. above normal, when running at the rate of 100 rev. per min. An additional compressor of the same capacity was supplied on each side of the river, in compliance with the requirement for 25% excess capacity; the additional compressors had also high-pressure air cylinders which could be connected at will, and in which the pressure could be increased to 150 lb., and the air used to supply rock drills, grouting machines, etc. The entire combination on each side of the river, therefore, was rated at 25 000 cu. ft. of free air per minute, or a mean of 6 250 cu. ft. per heading. Its safe working capacity was not far from 20 000 cu. ft. per min.

The shields broke through rock surface in Tunnels *B*, *C*, and *D*, at Manhattan, in November and December, 1905. The consumption of air in the four tunnels soon exceeded 15 000 cu. ft. for 24 hours, and in Tunnel *D*, on several occasions, it exceeded 7 000 cu. ft. for a like period. Blows had become frequent, and it was evident that the air plant was inadequate for driving four tunnels at once in the open material east of the Manhattan rock. Work in Tunnel *A*, therefore, was not resumed, after the suspension on December 29th, for about ten months, and Tunnel *C* was also closed down for more than four months of the time between December, 1905, and July, 1906. During this period the capacity of the plant was increased from the rated 25 000 cu. ft. of free air per minute, to 35 000. In Tunnel *D* the material had gradually become firmer, with more clay and less escape of air, as the Blackwell's Island Reef was approached, and, at the end of the period, the rock surface was within 3 ft. of the top of the shield; in Tunnel *B*, the rock of the reef was still a little below the shield, but the overlying material contained a large proportion of clay and held air very well. Tunnel *C* was still in open material, but, with two lines safe and with the increased air plant, it was deemed best to resume work in Tunnel *A*, which was done on October 23d, 1906. Thenceforward work was continuous in all headings until the meeting points with the Long Island shields were reached.

This period, January to October, 1906, inclusive, was the most strenuous of the entire work, particularly the first six months. With one and, at times, two tunnels closed down, the consumption of air in the headings from Manhattan was an average of more than 20 000

cu. ft. per min. for periods of from 30 to 60 days; it was often more than 25 000 cu. ft. for 24 hours, with a maximum of nearly 29 000 cu. ft., and doubtless this was exceeded considerably for shorter periods. On several occasions the quantity supplied to a single tunnel averaged more than 15 000 cu. ft. per min. for 24 hours. The greatest averages for 24 hours were obtained later in Tunnel *A*, after the resumption of work there, and exceeded 19 000 cu. ft., but the conditions in the headings of the other lines were then so favorable that the work was carried on continuously in all.

The deficiency in the original plant at Manhattan was so marked, and the need of driving all headings from Long Island simultaneously so clear, that it was decided to increase the rated capacity of the Long Island compressor plant to 45 400 cu. ft. of free air per minute, which was 10 400 cu. ft. greater than the capacity of the Manhattan plant after the latter had been augmented.

The earth encountered on emerging from rock, when driving westward from Long Island, was far more compact and less permeable to air than on the Manhattan side, but for a distance of from 400 to 600 ft. immediately east of the reef, it was a clean open sand, and, while the shields were passing through this, the quantity of air supplied to the four headings seldom fell below 20 000 cu. ft. per min.; it was usually more than 25 000 cu. ft., with a recorded maximum of 33 400 cu. ft. Although this was greater than ever used on the Manhattan side, it was more uniformly distributed among the several headings, and in none equalled the maximum observed on the Manhattan side, the largest having been 12 700 cu. ft. per min. for 24 hours; it must be remembered, however, that at one time only two tunnels were in progress in the bad material in the tunnels from Manhattan.

From the foregoing experience, it would seem that the plant finally furnished at Long Island, having a rated capacity of 45 400 cu. ft. of free air per minute, would have been a reasonable compliance with the original actual needs on the Manhattan side and *vice versa;* the plant finally developed on the Manhattan side, having a rated capacity of 35 000 cu. ft. of free air per minute, would have sufficed for the Long Island side.

The total quantity of free air compressed for the supply of the working chambers of the tunnels and the Long Island caissons was

34 109 000 000 cu. ft., and, in addition, 10 615 000 000 cu. ft. were compressed to between 80 and 125 lb. for power purposes, of which at least 80% was exhausted in the compressed-air working chambers. The total supply of free air to each heading while under pressure, therefore, averaged about 3 550 cu. ft. per min.

The quantity of air escaping during a sudden blow-out is apparently much smaller than might be supposed. Investigation of a number of cases, showing large pressure losses combined with a long stretch of tunnel supplying a relatively large reservoir of air, disclosed that a maximum loss of about 220 000 cu. ft. of free air occurred in 10 min. This averages only a little more than 19 000 cu. ft. per min., the maximum recorded supply to one tunnel for a period of 24 hours. Of this quantity, however, probably from 30 to 40% escaped in the first 45 seconds, while the remainder was a more or less steady loss up to the time when the supply could be increased sufficiently to maintain the lowered pressure. Very few blows showed losses approaching this in quantity, but the inherent inaccuracy of the observations make the foregoing figures only roughly approximate.

Special Difficulties.

The most serious difficulties of the work came near the start. In Tunnel *D* blows and falls of sand from the face were frequent after soft ground was met in the top. About six weeks after entering the full sand face, and before the shutters had been installed, the shield showed a decided tendency to settle, carrying the tunnel lining down with it and resulting in a number of badly broken plates in the bottom of the rings. Notwithstanding the use of extremely high vertical leads,* the sand was so soft that the settlement of the shield continued for about fifteen rings, the maximum being nearly 9 in. below grade. The hydrostatic head at mid-height of the tunnel was 32½ lb., and the raising of the air pressure to 37 lb., as was done at this time, was attended with grave danger of serious blows, on account of the recent disturbance of the natural cover by the pulling and re-driving of piles in the reconstruction of the Long Island ferry slips directly above. It dried the face materially, however, and the shield began to rise again, and had practically regained the grade when the anticipated blow-outs

* The lead of the shield is the angular divergence of its axis from the axis of the tunnel, and, in this tunnel, was measured as the offset in 23 ft. It was called + when the shield was pointed upward from grade, and — when pointed downward.

FIG. 1.—TEMPORARY REINFORCEMENT OF BROKEN PLATES AND REMOVAL OF A PLATE IN SECTIONS.

FIG. 2.—HEAVY CAST-STEEL PATCH ATTACHED TO BENT SEGMENT OF CUTTING EDGE.

FIG. 3.—INFLOW OF SOFT CLAY THROUGH SHIELD.

FIG. 4.—REINFORCEMENT OF BROKEN PLATE WITH LONG BOLT AND TWISTED STEEL RODS.

occurred, culminating with the entrance of rip-rap from the river bed into the shield and the flooding of the tunnel with 4 ft. of sand and water at the forward end. The escape of air was very great, and, as a pressure of more than 28 lb. could not be maintained, the face was bulkheaded and the tunnel was shut down for three weeks in order to permit the river bed to consolidate.

This was the most serious difficulty encountered on any part of the work, and, coming at the very start, was exceedingly discouraging. During the shut-down the broken plates were reinforced temporarily with steel ribs and reinforced concrete (Fig. 1, Plate XCIII) which, on completion of the work, were replaced by cast-steel segments, as described elsewhere. Practically, no further movement of iron took place, and the loss of grade caused by the settlement of the shield, which was by far the largest that ever occurred in this work, was not sufficient to require a change in the designed grade or alignment of the track. Work was resumed with the shutters in use at the face as an aid to excavation. The features of extreme seriousness did not recur, but for two months the escape of air continued to be extremely large, an average of 15 000 cu. ft. per min. being required on many days during this period.

In Tunnel *B*, after passing out from under the bulkhead line, in April, 1906, the loss of air became very great, and blow-outs were of almost daily occurrence until the end of June. At the time of the blows the pressure in the tunnel would drop from 2 to 8 lb., and it generally took some hours to raise the pressure to what it was before the blow. During that time regular operations were interrupted. In the latter part of June a permit was obtained allowing the clay blanket to be increased in thickness up to a depth of water of 27 ft. at mean low tide. The additional blanket was deposited during the latter part of June and early in July, and almost entirely stopped the blows.

By the end of the month the natural clay, previously described, formed the greater portion of the face, and, from that time forward, played an important part in reducing the quantity of air required. During April and the early part of May the work was under the ferry racks of the Long Island Railroad. The blanket had to be placed by dumping the clay from wheel-barrows through holes in the decking.

In Tunnel *A* a bottom heading had been driven 23 ft. in advance of the face at the time work was stopped at the end of 1905. During

the ten months of inactivity the seams in the rock above opened. The rock surface was only from 2 to 4 ft. below the top of the cutting edge for a distance of about 60 ft. Over the rock there were large boulders embedded in sharp sand. It was an exceedingly difficult operation to remove the boulders and place the polings without starting a run. The open seams over the bottom heading also frequently caused trouble, as there were numerous slides of rock from the face which broke up the breasting and allowed the soft material from above to run into the shield. There were two runs of from 50 to 75 cu. yd. and many smaller ones.

Guiding the Shields.

Little difficulty was experienced at any time in driving the shield close to the desired line, but it was much harder to keep it on grade. In rock section, where the cradle could be set far enough in advance to become hard before the shield was shoved over it, there was no trouble whatever. Where the cradle could be placed only a very short time before it had to take the weight of the shield, the case was quite different. The shield had a tendency to settle at the cutting edge, and when once pointed downward it was extremely difficult to change its direction. It was generally accomplished by embedding railroad rails or heavy oak plank in the cradle on solid foundation. This often had to be repeated several times before it was successful. In soft ground it was much easier to change the direction of the shield, but, owing to the varying nature of the material, it was sometimes impossible to determine in advance how the shield should be pointed. It was found by experience at Manhattan that the iron lining remained in the best position in relation to grade when the underside of the bottom of the shield at the rear end was driven on grade of the bottom of the iron, but if the rate of progress was slow, it was better to drive the shield a little higher.

In the headings from Long Island, which, as a rule, were in soft ground, the cutting edges of the shields were kept from 4 to 8 in. higher, with respect to the grade line, than the tails. The shields would then usually move parallel to the grade line, though this was modified considerably by the way the mucking was done and by the stiffness of the ground at the bottom of the shield.

On the average, the shields were shoved by from ten to twelve of the bottom jacks, with a pressure of about 4 000 lb. per sq. in. The

jacks had 9-in. plungers, which made the average total force required to shove the shield 2 800 000 lb. In the soft ground, where shutters were used, all of the twenty-seven jacks were frequently used, and on several occasions the pressure exceeded 6 000 lb. per sq. in. With a unit pressure of 6 000 lb. per sq. in., the total pressure on the shield with all twenty-seven jacks in operation was 5 154 tons.

Injuries to Shields.

There were only two instances of damage to the essential structural features of the shields. The most serious was in Tunnel *D* where the cutting edge at the bottom of the shield was forced up a slightly sloping ledge of rock. A bow was formed in the steel casting which was markedly increased with the next few shoves. Work was suspended, and a heavy cast-steel patch, filling out the bow, was attached to the bent segments, as shown in Fig. 2, Plate XCIII. No further trouble was experienced with the deformed portion. The other instance was in Tunnel *B*, from Long Island, where a somewhat similar but less serious accident occurred and was treated in a like manner.

Bulkheads.—At Manhattan, bulkheads had to be built near the shafts before the tunnels could be put under pressure. After 500 ft. of tunnel had been built on each line, the second bulkheads were constructed. The air pressure between the first and second bulkheads was then reduced to between 15 and 20 lb. When the shields had been advanced for 1 500 ft., the third set of bulkheads was built. Nearly all the broken plates which were removed were located between the first and third bulkheads at Manhattan. Before undertaking this operation, the doors of the locks in the No. 3 bulkheads were reversed to take pressure from the west. By this means it was possible to carry on the work of dismantling the shields under comparatively low pressure simultaneously with the removal of the broken plates.

At Long Island City the roofs of the caissons served the purpose of the No. 1 bulkheads. Two other sets of bulkheads were erected, the first about 500 ft. and the second about 1 500 ft. from the shafts.

Settlement at Surface of Ground.

The driving of such portions of the river tunnels, with earth top, as were under the land section, caused a settlement at the surface varying usually from 3 to 6 in. The three-story brick building at

No. 412 East 34th Street required extensive repairs. This building stood over the section of part earth and part rock excavation where the tunnels broke out from the Manhattan ledge and where there were a number of runs of sand into the shield. In fact, the voids made by those runs eventually worked up to the surface and caused the pavement of the alley between the buildings to drop 4 or 5 ft. over a considerable area. The tunnels also passed directly under the ferry bridges and racks of the Long Island Railroad at East 34th Street. Tunnels *B* and *D* were constantly blowing at the time, and, where progress was slow, caused so much settlement that one of the racks had to be rebuilt. Tunnel *A*, on the other hand, where progress was rapid, caused practically no settlement in the racks.

Clay Blanket.

As previously mentioned, clay was dumped over the tunnels in varying depths at different times. A material was required which would pack into a compact mass and would not readily erode under the influence of the tidal currents of the river and the escape of the great volumes of air which often kept the water in the vicinity of the shields in violent motion. Suitable clay could not be found in the immediate vicinity of the work. Materials from Shooter's Island and from Haverstraw were tried for the purpose. The Government authorities did not approve of the former, and the greater portion of that used came from the latter point. Although a number of different permits governing the work were granted, there were three important ones. The first permit allowed a blanket which roughly followed the profile of the tunnels, with an average thickness of 10 ft. on the Manhattan side and somewhat less on the Long Island City side. The second general permit allowed the blanket to be built up to a plane 27 ft. below low water. This proved effective in checking the tendency to blow, but allowed considerable loss of air. Finally, dumping was allowed over limited and marked areas up to a plane of 20 ft. below low water. Wherever advantage was taken of this last authority, the excessive loss of air was almost entirely stopped. After all the shields had been well advanced out into the river, the blanket behind them was dredged up, and the clay used over again in advance of the shield.

Soundings were taken daily over the shields, and, if marked erosion was found, clay was dumped into the hole. Whenever a serious blow

occurred, a scowload of clay was dumped over it as soon as possible and without waiting to make soundings. For the latter purposes a considerable quantity of clay was placed in storage in the Pidgeon Street slip at Long Island City, and one or two bottom-dump scows were kept filled ready for emergencies. Mr. Robert Chalmers, who had charge of the soundings for the contractor, states that "the depressions in the blanket caused by erosion due to the escape of air were, as a rule, roughly circular in plan and of a curved section somewhat flat in the center." Satisfactory soundings were never obtained in the center of a violent blow, but the following instance illustrates in a measure what occurred. Over Tunnel *B*, at Station 102+80, there was normally 36 ft. of water, 7 ft. of clay blanket, and 20 ft. of natural cover. Air was escaping at the rate of about 10 000 cu. ft. per min., and small blows were occurring once or twice daily. On June 22d, soundings showed 54 ft. of water. A depth of 18 ft. of the river bottom had been eroded in about two days. On the next day there were taken out of the shield boulders which had almost certainly been deposited on the natural river bed. Clay from the blanket also came into the shields on a number of occasions during or after blows. The most notable occasion was in September, 1907, when the top of the shield in Tunnel *D* was emerging from the east side of Blackwell's Island Reef. The sand in the top was very coarse and loose, and allowed the air to escape very freely. The fall of a piece of loose rock from under the breast precipitated a run of sand which was followed by clay from the blanket, which, in this locality, was largely the softer redredged material. Mucking out the shield was in progress when the soft clay started flowing again and forced its way back into the tunnel for a distance of 20 ft., as shown in Fig. 3, Plate XCIII. Ten days of careful and arduous work were required to regain control of the face and complete the shove, on account of the heavy pressure of the plastic clay.

The clay blanket was of the utmost importance to the work throughout, and it is difficult to see how the tunnels could have been driven through the soft material on the Manhattan side without it.

The new material used in the blanket amounted to 283 412 cu. yd., of which 117 846 cu. yd. were removed from over the completed tunnels and redeposited in the blanket in advance of the shields. A total of 88 059 cu. yd. of clay was dumped over blows. The total cost of placing and removing the blanket was $304 056.

Iron Lining.

The standard cast-iron tunnel lining was of the usual tube type, 23 ft. in outside diameter. The rings were 30 in. wide, and were composed of eleven segments and a key. The webs of the segments were 1½ in. thick in the central portion, increasing to 2⅜ in. at the roots of the flanges, which were 11 in. deep, 2¼ in. thick at the root, and 1½ in. at the edge, and were machined on all contact faces. Recesses were cast in the edge of the flanges, forming a groove, when the lining was in place, 1½ in. deep and about ⅜ in. wide, to receive the caulking. The bolt holes were cored in the flanges, and the bosses facing the holes were not machined. The customary grout hole was tapped in the center of each plate for a standard 1¼-in. pipe. In this work, experience indicated that the standard pipe thread was too fine, and that the taper was objectionable. Each segment weighed, approximately, 2 020 lb., and the key weighed 520 lb., the total weight being 9 102 lb. per lin. ft. of tunnel. Fig. 1 shows the details of the standard heavy lining.

In addition to the standard cast-iron lining, cast-steel rings of the same dimensions were provided for use in a short stretch of the tunnel, when passing from a rock to a soft ground foundation, where it was anticipated that unequal settlement and consequent distortion and increase in stress might occur, but, aside from the small regular drop of the lining as it passed out of the tail of the shield, no such settlement was observed.

Two classes of lighter iron, one with 1-in. web and 8-in. flanges and the other with 1¼-in. web and 9-in. flanges—the former weighing 5 166 lb. per lin. ft. of tunnel and the latter, 6 776 lb.—were provided for use in the land sections between East Avenue and the Long Island City shafts. Two weights of extra heavy segments for use at the bottom of the rings were also furnished. The so-called *XX* plates had webs and flanges ¼ in. thicker than the standard segment and the *YY* plates were similarly ½ in. heavier. The conditions under which they were used will be referred to later. All the castings were of the same general type as shown by Fig. 1.

Rings tapering ¾ in. and 1½ in. in width were used for changes in alignment and grade, the former being used approximately at every fourth ring on the 1° 30′ curves. The 1½-in. tapers were largely used for changes in grade where it was desired to free the iron from binding

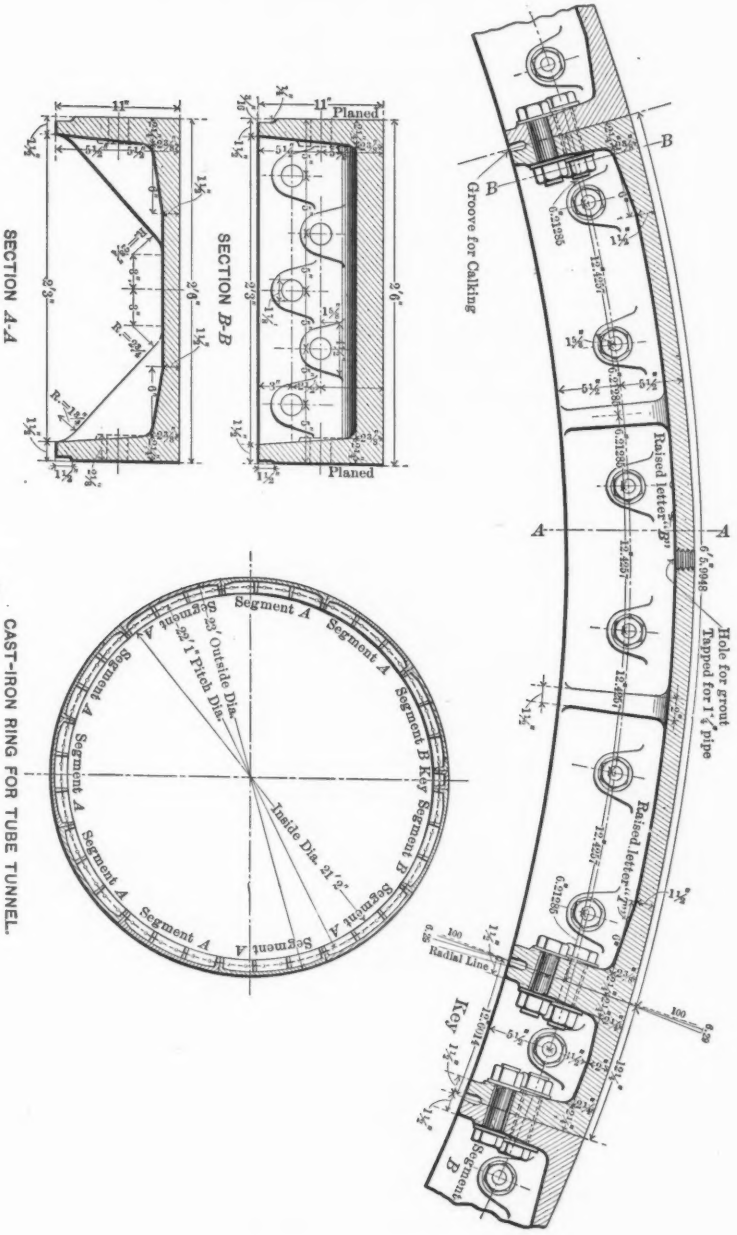


FIG. 1. CAST-IRON RING FOR TUBE TUNNEL.

on the tail of the shield. Still wider tapers would have been advantageous for quick results in this respect.

No lug was cast on the segments for attachment to the erector, but in its place the gadget shown on Fig. 4, Plate XC, was inserted in one of the pairs of bolt holes near the center of the plate, and was held in position by the running nut at one end.

In the beginning it was expected that the natural shape of the rings would not show more than 1 in. of shortening of the vertical diameter; this was slightly exceeded, however, the average distortion throughout the tunnels being $1\frac{7}{16}$ in. The erectors were attached to the shield and in such a position that they were in the plane of the center of the ring to be erected when the shove was made without lead and just far enough to permit placing the segments. If the shield were shoved too far, a rare occurrence, the erection was inconvenienced. In driving with high vertical leads, which occurred more frequently, the disadvantage of placing the erector on the shield was more apparent. Under such conditions the plane of the erector's motion was acutely inclined to the plane of the ring, and, after placing the lower portion of the ring, it was usually necessary to shove the shield a few inches farther in order to place the upper plates. The practical effect of this action is referred to later.

At first the erection of the iron in the river tunnels interfered somewhat with the mucking operations, but the length of time required to complete the latter was ample for the completion of the former; and the starting of a shove was seldom postponed by reason of the noncompletion of a ring. After the removal of the bottom of the diaphragms, permitting the muck cars to be run into the shield and beyond, the two operations were carried on simultaneously without serious interference. The installation of the belt conveyor for handling the soft ground spoil in Tunnel *A* was of special benefit in this respect.

Preparatory to the final bolt tightening of each ring as erected, a 15-ton draw-jack, consisting of a small pulling-jack inserted in a light eye-bar chain, was placed on the horizontal diameter, and frequently the erectors were also used to boost the crown of the iron, the object being to erect the ring truly circular. Before shoving, a $1\frac{1}{4}$-in. turnbuckle was also placed on the horizontal diameter in order to prevent the spreading of the iron, previous to filling the void outside with grout.

The approach of the supports for the upper floor of the trailing platform necessitated the removal of these turnbuckles from all but the three leading rings, but if the iron showed a tendency to continue distortion, they were re-inserted after the passage of the trailing platform and remained until the arch of the concrete lining was placed.

The cost of handling and erecting the iron varied greatly at different times, averaging, for the river tunnels, $3.32 per ton for the directly chargeable labor of handling and erecting, to which must be added $7.54 for "top charges." The cost of repairing broken plates is included in this figure.

Broken Plates.—During the construction of the river section of the tunnels, a number of segments were found to have been broken while shoving the shield. The breaks, which with few exceptions were confined to the three or four bottom plates, almost invariably occurred on the advanced face of the ring, and rarely extended beyond the bottom of the flange. A careful study of the breaks and of the shoving records disclosed several distinct types of fracture and three principal known causes of breakage by the shield.

In the first case, the accidental intrusion of foreign material between the jack head and the iron caused the jack to take its bearings on the flange above its normal position opposite the web of the ring, and resulted usually in the breaking out of a piece of the flange or in several radiating cracks with or without a depression of the flange. These breaks were very characteristic, and the cause was readily recognizable, even though the intruding substance was not actually observed.

In the second case, the working of a hard piece of metal, such as a small tool, into the annular space between the iron and the tail of the shield, where it was caught on the bead and dragged along as the shield advanced, was the known cause of a number of broken segments. Such breaks had no particular characteristic, but were usually close above the line of travel of the lost tool or metal. Their cause was determined by the finding of a heavy score on the underside of the segment or the discovery of the tool wedged in the tail of the shield or lying under the broken plate when it was removed. It is probable that a number of breaks ascribed to unknown causes should be placed in this class.

The third cause includes the largest number of breaks, and, while

difficult to define closely, is the most interesting. Broadly speaking, the breaks resulted from the movements of the shield in relation to the position of the tunnel lining. While shoving through soft ground, it was frequently difficult to apply sufficient power to the lower jacks to complete the full shove of 30 in. on the desired alignment. The shield, therefore, was driven upward at the beginning of the shove, and, as the sand packed in front of the shield and more power was required, it was furnished by applying the upper jacks. The top of the shield was slowly pushed over, and, at the close of the shove, the desired position had been obtained; but the shield had been given a rocking motion with a decided lifting of the tail toward the close of the shove. A similar lifting of the tail occurred when, with high vertical leads, the top of the shield was pushed over in order to place the upper plates of the ring. Again, when the shield was driven above grade and it was desired to descend, the passage of the shield over the summit produced a like effect. In all these movements, with the space between the tail of the shield and the iron packed tight with pugging, the upward thrust of the shield tended to flatten the iron in the bottom and occasional broken plates were the result. The free use of the taper rings, placed so as to relieve the binding of the lining on the tail of the shield, forces the tunnel to follow the variations in the grade of the shield, but reduces greatly the injuries to the rings from this action.

In Tunnel *D*, where very high vertical leads were required through the soft sand, combined with a marked tendency of the shield to settle, the shield was badly cramped on the iron and dragged along it at the top. The bearing of the iron on its soft foundation tended to thrust up the bottom in this case also, as shown by the opening of the bottom cross-joints when the bolts were slackened to relieve the strain during a shove. The anticipated cracks in the crown plates, which have been more frequently observed in other tunnels, did not occur here, and were not found elsewhere except in one place in Tunnel *B* where they were traced to a similar action of the shield. The cracks resulting from the movements of the shield, as briefly described above, in this third case were not confined to any particular type, but occurred more frequently at the extreme end of the circumferential flange than at any other point.

The number of broken plates occurring in the river tunnels was 319, or 0.42% of the total number erected. Of these, 52 were found and removed, either before or immediately after a shove, by far the

greater number being broken in handling before or during erection. The remaining 267 are considered below.

Repair of Broken Plates.—On the completion of a shove, the tail of the shield lacked about 5 in. of covering the full width of the last ring, and the removal of a plate broken during the shove, therefore, would have exposed the ground at the tail of the shield. With a firm material in the bottom, this introduced no particular difficulties, and, under such conditions, a broken plate was usually removed at once. In the sand, however, and especially on the Manhattan side where it was quick and flowing, the removal of a plate was attended with some danger, and such plates were usually left to be removed on the completion of the tunnel. Many of these had been reinforced by the use of *XX*, *YY*, and steel segments placed adjacent to the break in the following rings.

After the meeting of the shields, the postponed replacement of the broken segments was taken up. The pressure was raised sufficiently to dry thoroughly the sand outside the segments, which were drilled and broken out usually in quarters as shown on Fig. 1, Plate XCIII. A steel segment was then inserted in the ring and drawn into place by turnbuckles. The application of the draw-jack, with a pull of about 30 tons to each end successively, brought the plate to a firm bearing on the radial joints at the ends.

Where the broken plate was isolated and was reinforced by steel or extra heavy segments in the adjacent ring, the crack, if slight, was simply caulked to insure water-tightness. If, however, the crack was opened or extended to the web of the plate, the cross-flanges were tied together by a 1½-in. by 7-ft. bolt, inserted through the bolt holes nearest the broken flange. The long bolt acted in the nature of a bow string, and was provided at its ends with two nuts set on opposite sides of the cross-joints to replace the standard bolts removed for its insertion. Fig. 4, Plate XCIII shows one of these bolts in place. In addition, all broken plates remaining in the tunnel were reinforced with 1-in. twisted-steel rods in the concrete lining, also shown in Fig. 4, Plate XCIII.

Special Construction at River Shield Junctions.—Dismantling the shields was started as soon as they came to rest in their final position with the cutting edges together. The plans contemplated their entire removal, with the exception of the cylindrical skins and cast-steel

cutting edges. Inside the former the standard tunnel lining was erected to within 4 ft. of the heels of the cutting edges. Spanning the latter, and forming the continuous metal tunnel lining, the special construction shown by Fig. 2 was built. This consisted of a 1¼ in. rolled-steel ring, 7 ft. long, erected inside the cutting edges, with an annular clearance of 1 in., and two special cast-iron rings shaped to connect the rolled-steel ring with the normal lining. One flange of the special cast-iron rings was of the standard type, the other was returned 9 in. in the form of a ring, the inside diameter of which was the same as the outside diameter of the rolled-steel ring to which it was bolted.

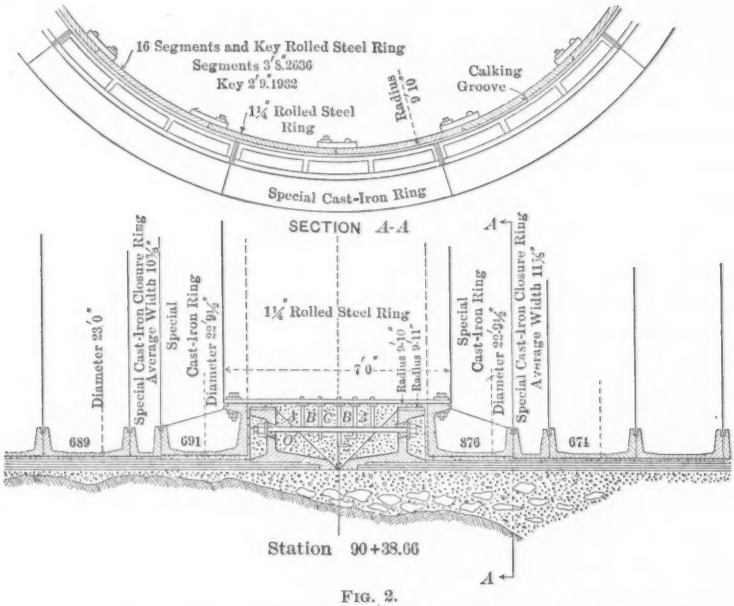


FIG. 2.

The space between the standard and special construction was of varying width at the various shields, and was filled with a closure ring cast to the lengths determined in the field. Fig. 2 shows the completed construction.

Hook-bolts, screwed through threaded holes and buried in 1 to 1 Portland cement grout ejected through similar holes, reinforced the rolled-steel ring against external water pressure. In two of the tunnels the concrete lining was carried completely through the junction, and

covered the whole construction, while in the remaining two tunnels it was omitted at the rolled-steel ring, leaving the latter exposed and set back about 3 in. from the face of the concrete.

Grouting.

Except as previously noted, the voids outside of the tunnel lining were filled with grout ejected through the grout holes in each segment. The possibility was always present that Portland cement, if used for grout in the shield-driven tunnels, would flow forward around the shield and set hard, "freezing" the shield to the rock or the iron lining, or at least forming excrescences upon it, which would render its control difficult. With this in mind, the contractors proposed to substitute an English Blue Lias lime as a grouting material. Grout of fresh English lime containing a moderate quantity of water set very rapidly in air to the consistency of chalk. Its hydraulic properties, however, were feeble, and in the presence of an excess of water it remained at the consistency of soft mud. It was not suitable, therefore, as a supporting material for the tunnel.

An American lime, made in imitation of the Lias lime, but having greater hydraulic properties, was tried, but proved unsatisfactory. Two brands of natural cement were also tried and rejected, but a modified quick-setting natural cement, manufactured especially for this work, was eventually made satisfactory, and by far the largest part of the river-tunnel grouting was done with this material mixed 1 to 1 by volume. East of the Long Island shafts the work which was built without shields was grouted principally with Portland cement and sand mixed 1 to 1 by volume.

In the river tunnels large quantities of the English lime were used neat as grout over the top of the tunnel in attempts to stop losses of air through the soft ground. It was not of great efficiency, however, in this respect until the voids outside of the lining had been filled above the crown. Its properties of swelling and quick setting in the dry sand at that point then became of value. The use of dry lime in the face, where the escaping air would carry it into the voids of the sand and choke them, was much more promptly efficacious in checking the loss.

With the exception of the English lime, all grout was mixed 1 to 1 with sand in a Cockburn continuous-stirring machine operated by a 3-cylinder air engine. The grout machine was placed on the lower floor

of the trailing platform shown on Plate XCII, while the materials were placed on the upper platform, and, together with the water, were fed into the machine through a hole in the upper floor. The sand was bagged in the yard, and the cars on which the materials were sent into the tunnels were lifted by an elevator to the level of the upper floor of the trailing platform before unloading.

Great difficulty was experienced in preventing the waste of the fluid grout ahead of the shield and into the tail through the space between it and the iron lining. In a full soft ground section, the first condition did not usually arise. In the full-rock sections the most efficient method of checking the waste was found to be the construction of dams or bulkheads outside the lining between it and the rock surface. For this purpose, at intervals of about 30 ft., the leading ring and the upper half of the preceding one were disconnected and pulled forward sufficiently to give access to the exterior. A rough dam of rubble, or bags of mortar or clay, was then constructed outside the iron, and the rings were shoved back and connected up. In sections containing both rock and soft ground, grout dams were built at the cutting edge at intervals, and were carried up as high as circumstances permitted.

The annular space at the tail of the shield was at all times supposed to be packed tight with clay and empty bags, but the pugging was difficult to maintain against the pressure of the grout. For a time, ½-in. segmental steel plates, slipped down between the jackets and the iron, were used to retain the pugging, but their displacement resulted in a number of broken flanges, and their use was abandoned. In their place, 2-in. segmental plates attached to the jack heads were substituted with more satisfactory results. Notwithstanding these devices, the waste of grout at the tail was very great.

The soft ground material on various portions of the work acted very differently. The clay and "bull's liver" did not cave in upon the iron lining for several hours after the shield had passed, sometimes not for a day or more, which permitted the space between it and the iron to be grouted. The fine gray or beach sand and the quicksand closed in almost at once. The quicksand has a tendency to fill in under the iron from the sides and in places to leave a cavity at about the horizontal diameter which was not filled from above, as the sand, being dried out by the air, stood up fairly well and did not cave against the iron, except where nearly horizontal at the top.

The total quantity of grout used on the work was equivalent in set volume to 249 647 bbl. of 1 to 1 Portland cement grout, of which 233 647 bbl. were ejected through the iron lining, an average of 14.93 bbl. per lin. ft. The cost of grout ejected outside of the river tunnels was 93 cents per bbl. for labor and $2.77 for "top charges." East of the Long Island shaft the corresponding costs were $0.68 and $1.63, the difference being partly due to the large percentages of work done in the normal air at the latter place.

Caulking and Leakage.

Up to August, 1907, the joints between the segments of the cast-iron lining were caulked with iron filings and sal ammoniac, mixed in the proportion of 400 to 1 by weight. With the air pressure balancing the hydrostatic head near the tunnel axis, it was difficult to make the rust-joint caulking tight below the axis against the opposing water pressure; this form of caulking was also injured in many places by water dripping from service pipes attached to the tunnel lining. A few trials of lead wire caulked cold gave such satisfactory results that it was adopted as a substitute. Pneumatic hammers were used successfully on the lead caulking, but were only used to a small extent on the rust borings, which were mostly hand caulked. Immediately before placing the concrete lining, all leaks, whether in the rust borings or lead, were repaired with lead, and the remainder of the groove was filled with 1 to 1 Portland cement mortar, leaving the joints absolutely water-tight at that time. The subsequent development of small seepages through the concrete would seem to indicate that the repair work should have been carried on far enough in advance of the concreting to permit the detection of secondary leaks which might develop slowly. The average labor cost chargeable against the caulking was 12 cents per lin. ft., to which should be added 21.8 cents for "top charges."

Unfortunately, it was necessary to place the greater part of the concrete lining in the river tunnels during the summer months when the temperature at the point of work frequently exceeded 85°; and the temperature of the concrete while setting was much higher. This abnormal heat, due to chemical action in the cement, soon passed away, and, with the approach of winter, the contraction of the concrete resulted in transverse cracks. By the middle of the winter these had developed quite uniformly at the ends of each 30-ft. section of con-

crete arch as placed, and frequently finer cracks showed at about the center of each 30-ft. section.

While the temperature of the concrete was falling, a like change was taking place in the cast-iron lining, with resulting contraction. The lining had been erected in compressed air, the temperature of which averaged about 70° in winter and higher in summer. Compressed air having been taken off in the summer of 1908, the tunnels then acquired the lower temperature of the surrounding earth, slowly falling until mid-winter. The contraction of the concrete, firmly bedded around the flanges of the iron, and showing cracks at fairly uniform intervals, probably localized the small corresponding movements of the iron near the concrete cracks, and resulted in a loosening of the caulking at these points. With the advent of cold weather, damp spots appeared in numerous places on the concrete, and small seepages showed through quite regularly at the temperature cracks, in some cases developing sufficiently to be called leaks. Only a few, however, were measurable in amount.

Early in January small brass plugs were firmly set on opposite sides of a large number of cracks, and caliper readings and air temperature observations were taken regularly throughout the winter and spring. The widths of the cracks and the amount of leakage at them increased with each drop in temperature and decreased as the temperature rose again, but until spring the width of the cracks did not return to the same point with each return of temperature.

The leakage was similar in all four tunnels, but was largest in amount in Tunnel *D*, where, at the beginning of February, the ordinary flow was about 0.0097 cu. ft. per sec., equivalent to 0.00000347 cu. ft. per sec. per lin. ft. of tunnel. Of this amount 0.0065 cu. ft. per sec. could be accounted for at eight of the cracks showing measurable leakage, leaving 0.0032 cu. ft. per sec. or 0.00000081 cu. ft. per sec. per lin. ft. of tunnel to be accounted for as general seepage distributed over the whole length.

It was not feasible to stop every leak in the tunnel, most of which were indicated simply by damp spots on the concrete; a rather simple method was devised, however, for stopping the leaks at the eight or ten places in each tunnel where water dripped from the arch or flowed down the face of the concrete. The worst leak in any tunnel flowed about 0.0023 cu. ft. per sec. To stop these leaks, rows of 1-in. holes,

at about 4-in. centers, were drilled with jap drills through the concrete to the flange of the iron. These rows were from 3 to 18 ft. long, extending 1 ft. or more beyond the limits of the leak. The bottoms of the holes were directly on the caulking groove and the pounding of the drill usually drove the caulking back, so that the leak became dry or nearly so after the holes were drilled. If left alone the leaks would gradually break out again in a few hours or a few days and flow more water than before. They were allowed to do this, however, in only a few cases as experiments. After the holes were drilled, the bottom 4 in. next the flange was filled with soft neat cement mortar. Immediately on top of this was placed two plugs of neat cement about 2½ in. long, which were 5 or 6 hours old and rather hard. Each was tamped in with a round caulking tool of the size of the hole driven with a sledge hammer. On top of this were driven in the same way two more plugs of neat cement of the same size, which were hard set. These broke up under the blows of the hammer, and caulked the hole tight. When finished, the tamping tool would ring as though it was in solid rock. Great pressure was exerted on the plastic mortar in the bottom of the hole, which resulted in the re-caulking of the joint of the iron. No further measurable leakage developed in the repaired cracks, during a period of four months, and the total leakage has been reduced to about 0.002 cu. ft. per sec. in each tunnel, an average of 0.00000051 cu. ft. per sec. per lin. ft.

Sump and Pump Chambers.

To take care of the drainage of the tunnels, a sump with a pump chamber above it was provided for each pair of tunnels. The sumps were really short tunnels underneath the main ones and extending approximately between the center lines of the latter. They were 10 ft. 9½ in. in outside diameter and 44 ft. long. The water drops directly from the drains in the center lines of the tunnels into the sumps. Above the sumps and between the tunnels, a pump chamber 19 ft. 5 in. long was built. Above the end of the latter, opposite the sump, a cross-passage was constructed between the bench walls of the two tunnels. This passage gives access from either tunnel through an opening in the floor to the pump chamber and through the latter to the sump.

From the preliminary borings it was thought that the sumps were

located so that the entire construction would be in rock. This proved to be the case on Tunnels *C* and *D*, but not on Tunnels *A* and *B*. The position of the rock surface in the latter is shown by Fig. 3. After the excavation was completed in Tunnel *B*, January 1st, 1908, the plates were removed from the side of the tunnel at the cross-passage, and a drift was driven through the earth above the rock surface across to the lining of Tunnel *A*. The heading was timbered as shown by Fig. 3. There was practically no loss of air from the drift, but the clay blanket had been removed from over this locality and the situation caused some anxiety. In order to make the heading as secure as possible, the 24-in. I-beams, shown on Fig. 3, were attached to the lining of the two tunnels. The beams formed a support for the permanent concrete roof arch of the passage, which was placed at once. At the same time plates were removed from the bottom in Tunnel *B* over the site of the sump, and a heading was started on the line of the sump toward Tunnel *A*. As soon as the heading had been driven beyond the center line of the pump chamber, a bottom heading was driven from a break-up westward in the pump chamber and a connection was made with the cross-passage. The iron lining of the pump chamber was next placed, from the cross-passage eastward. The soft ground was excavated directly in advance of the lining, and the ground was supported by polings in much the same manner as described for shield work. On account of bad ground and seams of sand encountered in the rock below the level of the cross-beams, the entire west wall of the pump chamber was placed before enlarging the sump to full size. This was also judicious, in order to support as far as possible the iron lining of the tunnels. The sump was then excavated to full size. The iron lining of the sump and the east wall of the pump chamber were placed as soon as possible. The voids outside the iron lining of the sump and the pump chamber were filled as completely as possible with concrete, and then thoroughly grouted. Finally, the concrete lining was put in place inside of the iron.

As shown by Fig. 3, the excavation of these chambers left a considerable portion of the iron lining of the tunnels temporarily unsupported on the lower inner quarter. To guard against distortion, a system of diagonals and struts was placed as shown.

The floor of the pump chamber was water-proofed with felt and pitch in a manner similar to that described for the caissons at Long

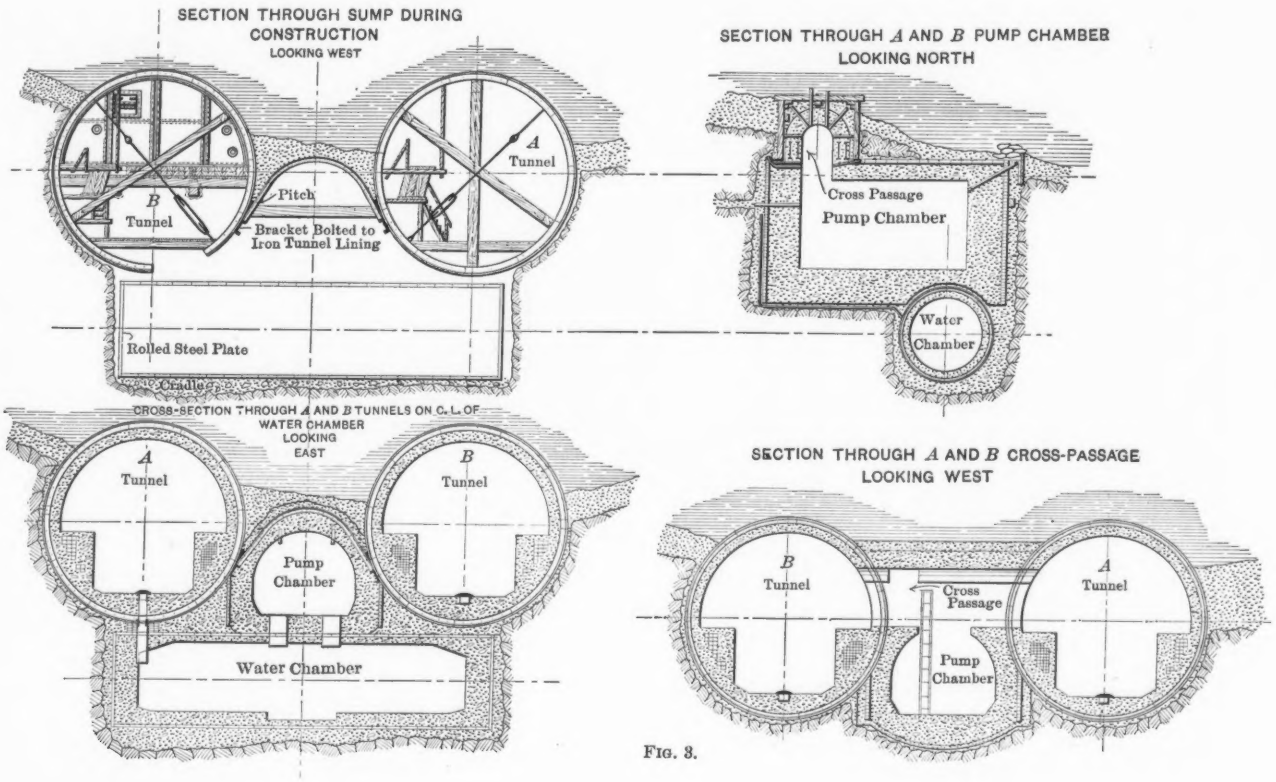

SECTION THROUGH SUMP DURING CONSTRUCTION
LOOKING WEST
A Tunnel
B Tunnel
Pitch
Bracket Bolted to Iron Tunnel Lining
Rolled Steel Plate
Cradle
SECTION THROUGH A AND B PUMP CHAMBER
LOOKING NORTH
Cross Passage
Pump Chamber
Water Chamber
CROSS-SECTION THROUGH A AND B TUNNELS ON C. L. OF WATER CHAMBER
LOOKING EAST
A Tunnel
B Tunnel
Pump Chamber
Water Chamber
SECTION THROUGH A AND B CROSS-PASSAGE
LOOKING WEST
B Tunnel
A Tunnel
Cross Passage
Pump Chamber

FIG. 3.

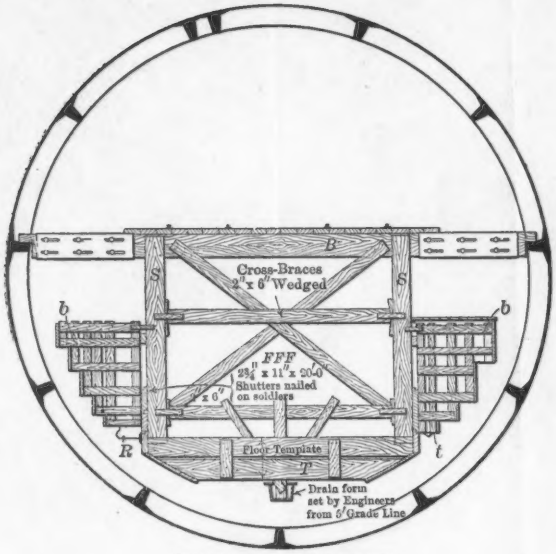
Island City. It was not possible to make the felt stick to the vertical walls with soft pitch, which was the only kind that could be used in compressed air, and, therefore, the surfaces were water-proofed by a wall of asphalt brick laid in pitch melting at 60° Fahr. Forms were erected on the neat line, and the space to the rock was filled with concrete making a so-called sand-wall similar to that commonly used for water-proofing with felt and pitch. The bricks were then laid to a height of four or five courses. The joints were filled with pitch instead of mortar. Sheets of tin were then placed against the face of the wall and braced from the concrete forms. As much pitch as possible was then slushed between the brick and the sand-wall, after which the concrete in the main wall was filled up to the top of the water-proofing course. The tin was then withdrawn and the operation repeated. This method was slow and expensive, but gave good results. Ordinary pitch could not be used on account of the fumes, which are particularly objectionable in compressed air. The 60° pitch was slightly heated in the open air before using.

The sump and pump chamber on Tunnels *C* and *D* differed from the one described only in minor details; but, being wholly constructed in rock, presented fewer difficulties and permitted a complete envelope of water-proofing to be placed in the top.

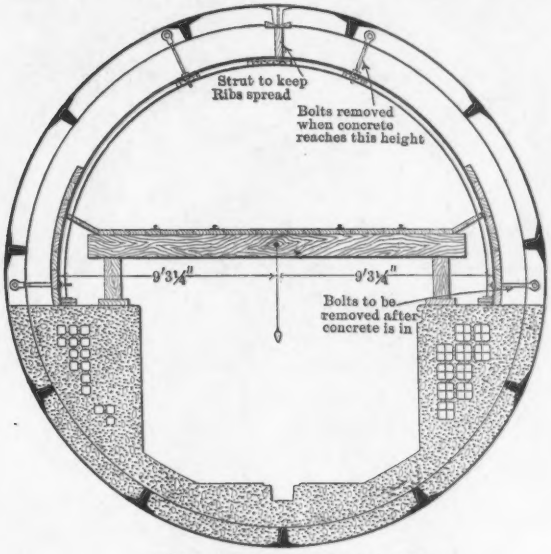
Concrete Lining.

The placing of concrete inside the iron tube was done by an organization entirely separate from the tunneling force. A mixing plant was placed in each of the five shafts. The stone and sand bins discharged directly into mixers below, which, in turn, discharged into steel side-dump concrete cars. All concrete was placed in normal air.

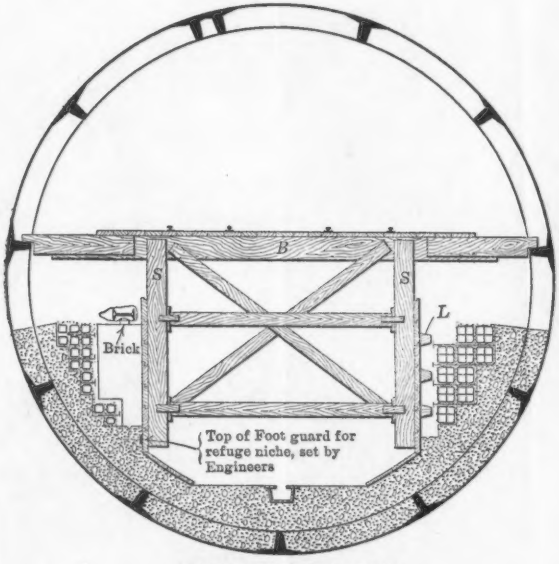
The first step, after the iron lining was scraped clean and washed down and all leaks were stopped, was the placing of biats, marked *B* on Plate XCIV. These were made up of a 6 by 12-in. yellow pine timber, 17 ft. long, with two short lengths of the same size spliced to its ends by pieces of 12-in. channels, 3 ft. 9 in. long, clamped upon the sides. These biats were placed every 5 ft. along the tunnel in rings having side keys. Next, a floor, 13 ft. wide, was laid on the biats and two tracks, of 30-in. gauge and 6½-ft. centers, were laid upon the floor. There were three stages in the concreting. Fig. 2, Plate XCIV, shows the concrete in place at the end of the first, and Fig. 3, Plate



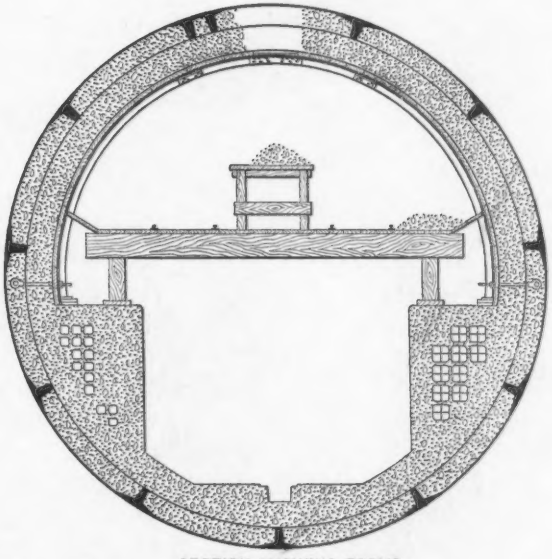
SECTION SHOWING FORMS
FOR INVERT
FIG. 1



SECTION SHOWING ARCH FORMS
READY FOR CONCRETE
FIG. 3



SECTION SHOWING FORMS
FOR BENCH
FIG. 2



SECTION SHOWING FORMS
FOR KEY
FIG. 4

XCIV, at the end of the second stage. The complete arch above the bench walls was done in the last operation.

Two 3 by 10-in. soldiers (*SS* in Figs. 1 and 2, Plate XCIV) were fastened to each biat and braced across by two horizontal and two diagonal braces. To each pair of soldiers a floor template, *T*, was then nailed. The form for the center drain was then suspended as shown in Fig. 1, Plate XCIV. Three pieces of shuttering, *FFF*, 20 ft. long, were then nailed to the bottom of the soldiers. One is all that would have been needed for the first concrete placed, but it was easier to place them at this stage than later, when there was less room. Three rough shutters were also nailed to the curved portion for the floor template. Opposite each biat, a bracket, *bb*, was then nailed, which carries a set of rough boards which formed the risers for the duct steps. Everything was then ready for concreting except that, where refuge niches occurred, a form for the portion of the niche below the seat was nailed to the shuttering. This form is shown at *R* in Fig. 1, Plate XCIV.

The concrete was dumped down on each side from side-dump cars standing on the track, and, falling between the risers for the duct steps, ran or was shoveled under the forms and down into the bottom. The horizontal surface on each side the center drain was smoothed off with a shovel. The workmen became very skillful at this, and got a fairly smooth surface. This concrete was usually placed in lengths of 45 or 60 ft. After setting for about 24 hours, the brackets, *bb*, were removed, together with the shuttering on the steps. The triangular pieces, *t* in Fig. 1, Plate XCIV, were not removed until later. Instead, a board was laid upon this lower step on which the duct layers could work. This and the triangular piece were not removed until just before the bench concrete was placed. This was important, as otherwise the bond between the old and new concrete would be much impaired by dirt ground into the surface of the old concrete. The ducts were then laid, as shown in Fig. 2, Plate XCIV.

The remaining shutters for the face of the bench walls were then placed. The remainder of the forms for the refuge niches, *RR*, in Fig. 1, Plate XCIV, were nailed to the shutters, the steel beam over the niche was laid in place, the forms for the ladders, *L* in Fig. 2, Plate XCIV, which occur every 25 ft., were tacked to the shutters, the shutters and forms were given a coat of creosote oil, and then all was ready for placing the bench concrete.

The specifications required a 2-in. mortar face to be placed on all exposed surfaces and the remainder to be smoothed with a trowel and straight-edge. After about 48 hours, the biats were blocked up on the bench, and all forms between the bench walls below the working floor were removed.

The centering for the arch concrete consisted of simple 5 by 3½ by $\frac{5}{16}$-in. steel-angle arch ribs, curved to the proper radius, spaced at 5-ft. intervals. Each rib was made up of two pieces spliced together at the top. Two men easily handled one of these pieces. After splicing, the rib was supported by four hanger-bolts fastened to the iron lining as shown in Fig. 3, Plate XCIV.

In the early part of the work, two additional bolts were used about half way up on the side between the upper and lower hanger-bolts. It was soon found that by placing the strut between the tunnel lining and the crown of the rib, these hanger-bolts could be dispensed with. The lagging was of 3-in. dressed yellow pine, 12 in. wide, and in 15-ft. lengths. Each piece had three saw cuts on the back, from end to end, allowing it to be bent to the curve of the arch; it was kept curved by an iron strap screwed to the back. The arches were put in, either in 15, 30 or 45-ft. lengths, depending on what was ready for concrete and what could be done in one continuous working. The rule was that when an arch was begun, the work must not stop until it was finished. An arch length always ended in the middle of a ring. The lagging was placed to a height of about 6 ft. above the bench before any concreting was done. When the concrete had been brought up to that point, lagging was added, one piece at a time, just ahead of the concrete, up to the crown, where a space of about 18 in. was left. When the lagging had reached the upper hanger-bolts, they were removed, which left only the two bottom bolts fixed in the concrete. Most of these were unscrewed from the eye and saved, as tin sleeves were placed around them before concreting. Two cast-iron eyes were lost for every 5 ft. of tunnel. To place the key concrete, a stage was set up in the middle of the floor, and, beginning at one end, about 2 ft. of block lagging was placed. Over this, concrete was packed, filling the key as completely as possible. This was done partly by shoveling and using a short rammer, and partly by packing with the hands by the workmen, who wore rubber gloves for the purpose. Another 2 ft. of lagging was then placed, and the operation was repeated, and thus working back-

ward, foot by foot, the key was completed. This is the usual way of keying a concrete arch, but in this case the difficulty was increased by the flanges of the iron lining. It was practically impossible to fill all parts of the pockets formed by these flanges. To meet this difficulty, provision was made for grouting any unfilled space. As the concrete was being put in, tin pipes were placed with their tops nearly touching the iron lining, and their bottoms resting on the lagging. Each pocket was intended to have two of these pipes, one to grout through and the other to act as a vent for the escape of air. Each center key ring had six pipes, and each side key had eight. The bottoms of the pipes were held by a single nail driven half way into the lagging. This served to keep the pipes in position and to locate them after the lagging was taken down.

The cost of labor in the tunnels directly chargeable to concrete was $1.80 per cu. yd. The top charges, exclusive of the cost of materials (cement, sand, and stone), amounted to $3.92.

Electric Conduits.

In one bench wall of each tunnel there were fifteen openings for power cables and in the other, between the river shafts, there were forty openings for telephone, telegraph, and signal cables. East of the Long Island shaft, the number of the latter was reduced to twenty-four. The telephone ducts were all of the four-way type. The specifications required that the power ducts should have an opening of not less than $3\frac{1}{2}$ in., nor more than $3\frac{3}{4}$ in., and that after laying they should pass a 4-ft. mandrel, $3\frac{3}{8}$ in. at the leading end and $2\frac{5}{8}$ in. at the other. The outside dimension was limited between 5 and $5\frac{3}{8}$ in. The openings of the four-way ducts were required to be not less than $3\frac{3}{4}$ in., nor more than $3\frac{5}{8}$ in., and after laying to pass a 5-ft. mandrel, $3\frac{1}{4}$ in. at the leading end and $2\frac{1}{2}$ in. at the other. The outside dimensions were limited between 9 and $9\frac{1}{2}$ in. All were to be laid in $\frac{1}{4}$-in. beds of mortar. The specifications were not definite as to the shape of the opening, but those used were square with corners rounded to a radius of $\frac{3}{8}$ in. The four-ways were 3 ft. long, and the singles, 18 in.

A study of the foregoing dimensions will show that the working limits were narrow. Such narrow limits would not pay for the ordinary conduit line in a street, where there is more room. In the tunnel greater liberality meant either reducing the number of conduits or

encroaching on the strength of the concrete tunnel lining. The small difference of only $\frac{1}{8}$ in. in the size of the mandrel, or a clearance of only $\frac{1}{16}$ in. on each side, no doubt did increase the cost of laying somewhat, though not as much as might at first be supposed. All bottom courses were laid to a string, in practically perfect line and grade, and all joints were tested with mandrels which were in all openings, and pulled forward as each piece of conduit was laid. As the workmen became skillful, the progress was excellent.

All costs of labor in the tunnel chargeable to duct laying amounted to $0.039 per ft. of duct; top charges brought this up to $0.083.

The serious problem was to guard against grout and mortar running into the duct opening through the joints from the concrete, which was a rather wet mixture. Each joint was wrapped, when laid, with canvas, weighing 10 oz. per sq. yd., dipped in cement grout immediately before using. These wraps were 6 in. wide, and were cut long enough to go around the lap about the middle of the duct. As soon as all the ducts were laid, the entire bank was plastered over with fairly stiff mortar, which, when properly done, closed all openings. The plastering was not required by the specifications, but was found by the contractor to result in a saving in ultimate cost.

The concrete on the two sides of the bank of ducts was bonded together by 2 by $\frac{1}{8}$-in. steel bonds between the ducts, laid across in horizontal joints. Both ends were split into two pieces, 1 in. long, one of which was turned up and the other down. These bonds projected $1\frac{1}{2}$ in. into the concrete on either side. Where the bond came opposite the risers of the duct step, against which the ducts were laid, recesses were provided for the projecting bond. This was done by nailing to the rough shutters for the steps a form which when removed left a dove-tailed vertical groove. This form was made in two pieces, one tapering inward and the other with more taper outward. As the bonds were placed, these grooves were filled with mortar.

The ducts usually received their final rodding with the specification mandrel a month or more after they were laid, after which all openings into splicing chambers were stopped by wooden plugs, 8 in. long tapering from $3\frac{3}{4}$ in. at one end to $2\frac{3}{4}$ in. at the other end, and shaped to fit the opening tightly. At first the plugs were paraffined, to keep them from swelling and breaking the ducts, but were not successful, as the paraffin lubricated them so that they would not stay in place. They

were expensive, and there was some swelling in the best that were obtained. A better plug was made by using no paraffin, but by making six saw cuts, three horizontal and three vertical, in the larger end, cutting to within about 2 in. of the smaller end. The swelling of the wood was then taken up by the saw cuts and the spring of the wood.

The splicing chambers are at 400-ft. intervals. They are 6 ft. long, 4 ft. 9 in. high, with a width varying from 3 ft. 2 in. at the top to 1 ft. 2 in. at the bottom.

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AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

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RIVER PROTECTION WORK ON THE KANSAS CITY SOUTHERN RAILWAY, NEAR BRADEN, OKLA.

By J. A. Lahmer, M. Am. Soc. C. E.

To be Presented November 3d, 1909.

When the branch of the Kansas City Southern Railway to Fort Smith, Ark., was built (in 1898), the track was about ¼ mile from the nearest point on the bank of the Arkansas River, near Braden, Okla. In recent years the river has been eroding its south bank at this place, so that in July, 1908, after the spring rise, it was 330 ft. from the track to the river. In the fall of that year there were unusual floods which caused the cutting of the bank to continue until the situation became very threatening, the river being within 180 ft. of the center of the track early in December, and erosion proceeding at a rapid rate. On account of topographic conditions it was not advisable to make a permanent change in the location of the track which, for some distance in this vicinity, is on the highest ground between the Arkansas and Poteau—one of its tributaries—which is a little less than a mile south of the track and nearly parallel to it.

The danger of a wash-out was averted temporarily by rolling into the river large cylindrical bundles of brush, 6 or 8 ft. in diameter and from 50 to 150 ft. long, one end of each bundle being anchored

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

to the track by cables. These bundles of brush contained a layer of sand bags, which caused the brush to sink. Next to the bank the current, of course, was very strong, so that the brush remained against the bank. In this way a wearing surface which the water would not cut was made from a line which was afterward found to be near low-water mark to a line a few feet above the flood stage, along that part of the bank which was so close as to endanger the track. All this work was done while the river was at flood stage. For several days it seemed that it would probably be necessary to move the track temporarily, but the brushwork proved equal to the emergency, and the river did not approach nearer than 50 ft. to the track.

Before the temporary work was completed, preparations were made for more durable protection work; in fact, plans had been adopted before the occurrence of the unexpected rise of December, 1908. The David Neale system of brush dikes had been selected, and bids were invited from contractors, but, on account of the scarcity of suitable material and the threatening condition of the river, no bids had been received. It was arranged, therefore, to do the work by Company forces under the direction of the Engineering Department.

The detailed plans of the dikes are shown on Plate XCV. Each dike consists essentially of a foundation of fascines, on which two or more stories or tiers of mud-cell rafts are built, as shown in the longitudinal section and plan, Figs. 1 and 2, Plate XCV.

The base of a fascine is made of a bottom layer of poles, *E*, from 4 to 6 in. in diameter and from 18 to 30 ft. long, on which there is a a layer of brush, *F*, 1 ft. thick. The brush and poles are laid longitudinally, with the butt ends of the brush outward at the ends of the fascine. Cross-walls, *G*, and side-walls, *H*, of brush are built on this floor in alternate courses from 10 to 12 in. thick, so that the fascine is divided into a series of cells from 6 to 8 ft. square and from 6 to 8 ft. deep. The length of the fascine was made equal to the width of the dike. The top of the fascine is a layer of brush, *I*, 1 ft. thick. The brush, in the walls and the top layer, is placed with the butt ends outward at the ends of the fascine.

An anchor of 6-in. poles, about 10 ft. long, is built in the second cross-wall from the up-stream end of the fascine. A $\frac{3}{4}$-in. or $\frac{7}{8}$-in. steel or iron cable (usually second-hand), secured to a post or tree on shore, is fastened to this anchor. Only every third or fourth fascine

is anchored to the shore in this way, the intermediate ones being tied by short cables to one of the cables which extends to the bank. Each cell is tied at the middle by a binder, *D*, made of three strands of No. 8 annealed wire. This binder is tightened by having its slack wound around a 4-in. stake, *J*. driven vertically through the center of each cell.

The first few fascines are built on the bank of the river and rolled into the water, until there are enough to form a space on which a fascine can be built and from which it can be rolled to the position it is to occupy in the dike. The work is continued until a foundation of the required width and length has been provided for the dike, the height of the foundation being such that the upper surface of the upper layer of fascines is above water level (Fig. 1, Plate XCV). Care is taken to place the fascine approximately parallel to the thread of the current, and to set each one a little down stream from the preceding one, in order to form an angle of approximately 45° between the thread of the current and the up-stream line of the fascine (Fig. 2, Plate XCV).

The lower mud-cell raft is started by laying rows of poles, from 4 to 6 in. thick, on top of the fascine foundation, lengthwise and crosswise of the dike, and from 6 to 8 ft. apart, as shown at *K* in Figs. 2 and 6, Plate XCV. The poles are wired together where they cross and are also wired to the binders of the upper course of fascines. At each intersection of the poles a stake, *L*, from 4 to 6 in. in diameter and from 8 to 10 ft. long, is nailed in a vertical position. A tie, of No. 8 annealed wire (Fig. 9, Plate XCV), is placed under the poles at each intersection and passed up along the stake, the ends of the tie extending about 1 ft. above the top of the stake. Walls of brush, *K*, are built in 10 or 12-in. courses along the rows of poles and on the up-stream and shore sides of the stakes, the courses being alternated lengthwise and crosswise of the dike until the walls are as high as the stakes. Fig. 8, Plate XCV, is a cross-section of the mud-cell raft.

The heights of the walls are regulated so that the finished dike at the outer end will be about 2 ft. lower on the up-stream side, *M*, than on the down-stream side, *N*, and, at the shore end of the dike, the up-stream side, *O*, will be about 6 ft. lower than the down-stream side, *P*, the object being to give the top of the dike a plow-share shape, in order to deflect the current as much as possible.

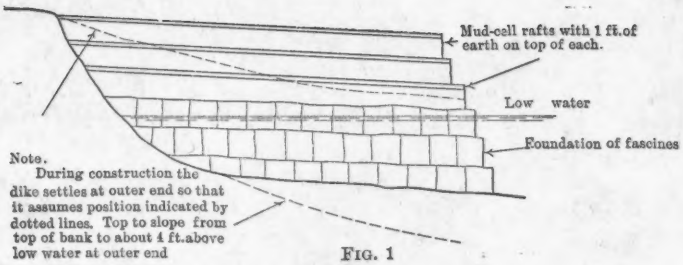


FIG. 1

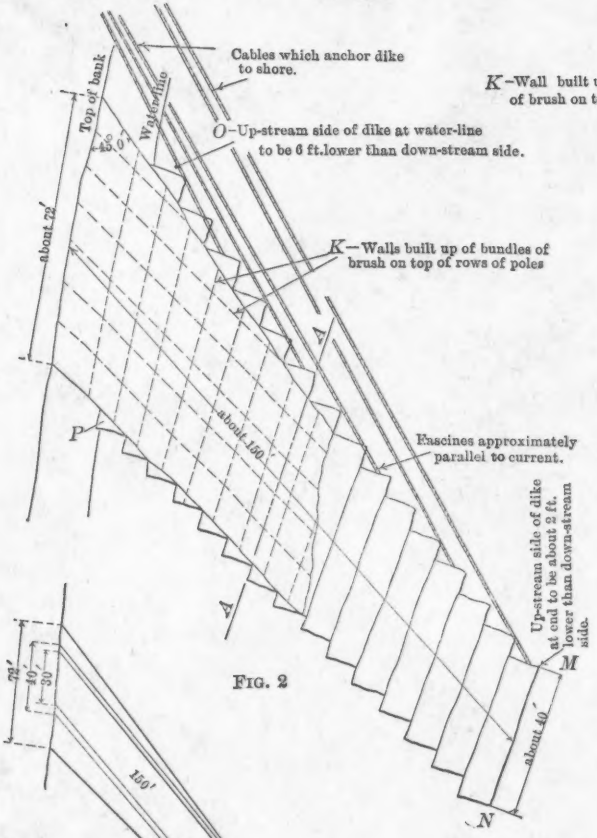


FIG. 2

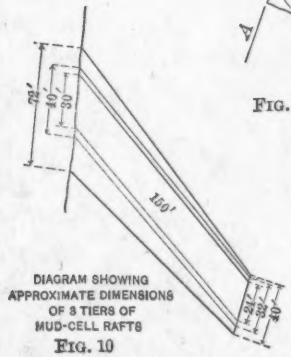


FIG. 10

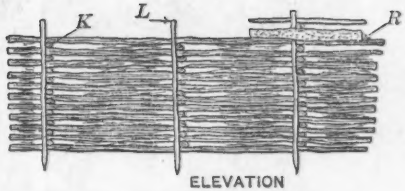
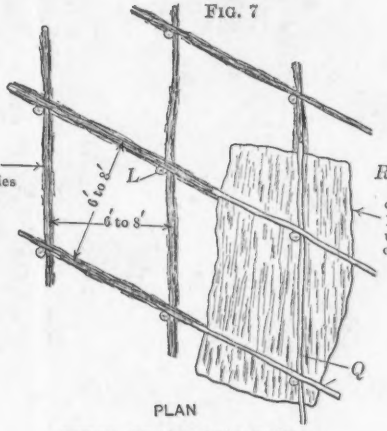


FIG. 7



DETAIL OF MUD-CELL RAFT

FIG. 6

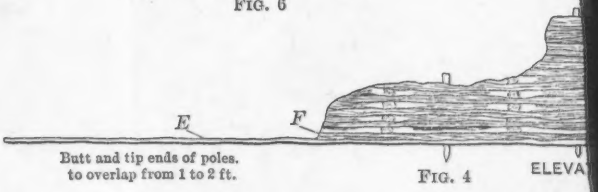
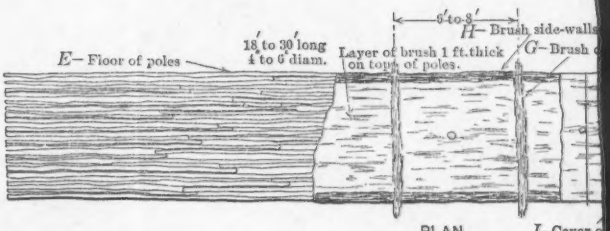


FIG. 4



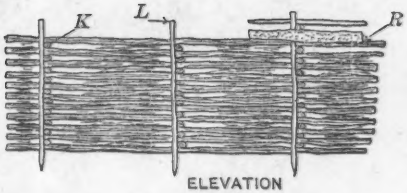
DETAIL OF FASCINE

FIG. 3

ll rafts with 1 ft. of
n top of each.

Low water

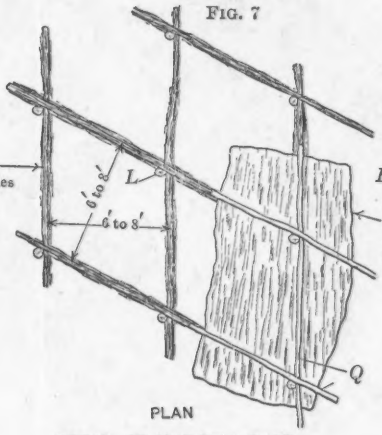
Foundation of fascines



ELEVATION

FIG. 7

all built up of bundles
brush on top of row of poles



Cover of brush
of poles on top
No. 8 annealed w
up the stakes at
cover.

PLAN

DETAIL OF MUD-CELL RAFT

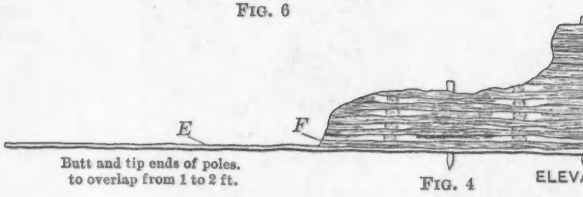
FIG. 6

nately
rent.

Up-stream side of dike
at end to be about 2 ft.
lower than down-stream
side.

M

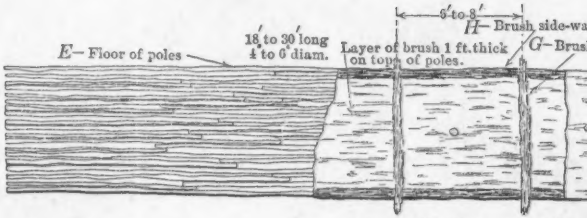
about 40′



Butt and tip ends of poles.
to overlap from 1 to 2 ft.

FIG. 4

ELEV



PLAN

I- Cove

DETAIL OF FASCINE

FIG. 3

PLATE XCV.
PAPERS, AM. SOC. C. E.
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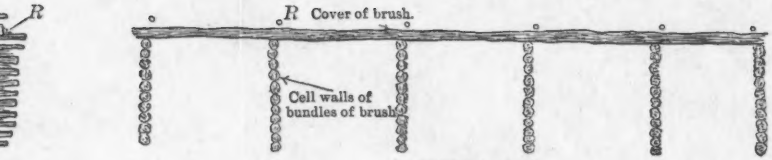
LAHMER ON
RIVER PROTECTION WORK.



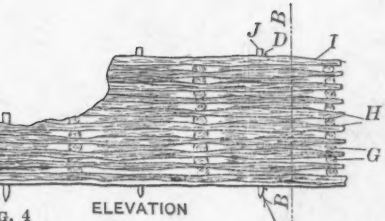
SECTION *A-A*

FIG. 8

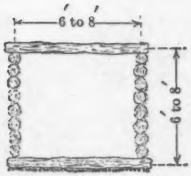
Wire around bottom pole fastened to stake with staples and two ends of wire extend up around top pole and are twisted together.

DIAGRAM SHOWING TIE WIRE
AT WALL INTERSECTIONS
OF MUD-CELL RAFTS

FIG. 9

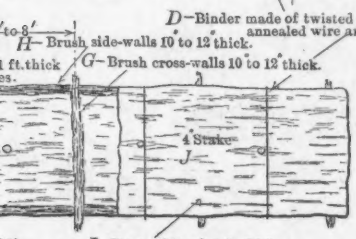


G. 4 ELEVATION



SECTION *B-B*

FIG. 5



D—Binder made of twisted No.8 annealed wire around each cell.

AN
F FASCINE

G. 3

PLAN OF BRUSH DIKES

AS CONSTRUCTED ON THE ARKANSAS RIVER
NEAR BRADEN, OKLA.
FOR THE KANSAS CITY, SOUTHERN RAILWAY

A cover of brush, *R*, 1 ft. thick, is built over the raft, the brush being placed approximately parallel to the thread of the current, with its butt ends outward at the up-stream and down-stream edges of the dike. On top of the brush cover, rows of poles, *Q*, are laid so as to intersect at the vertical stakes, the ends of the wires which were brought up the stakes being tied around the poles at the intersections, as shown in Fig. 9, Plate XCV. The wires are then twisted tight, so that the upper and lower framework of poles and the intervening brush will be bound together.

Each mud-cell raft is covered with a layer of earth, from 10 to 12 in. deep, care being taken to leave the poles exposed at their intersections. The wires which unite the poles at the bottom and top of the raft are again tightened after the earth has been deposited, as the weight of the earth will have compressed the brush.

The upper mud-cell rafts are built in manner similar to the lower one, except that the top of the lower raft is used as the bottom of the next raft above.

The lower mud-cell raft is built to cover as nearly as practicable the entire upper surface of the fascine foundation. The second and third stories of mud-cell rafts are narrower, the approximate dimensions of the three stories being shown in Fig. 10, Plate XCV. The dikes are built so that the outer end (usually about 150 ft. distant from the shore connection) will be 4 or 5 ft. above low-water mark, and the shore end either level with the bank or a few feet above high-water mark. The distance between the dikes is made as near 500 ft. as the condition of the bank will permit.

During a rise in the river, while sediment is being carried by the water, the dikes obstruct the flow, and, as the velocity is reduced, sand and mud are deposited in the hollow spaces of the fascines and mud-cell rafts. Sedimentation also takes place between the dikes, first on the up-stream side of the dike, near the river bank, and then on the down-stream side for a greater distance from the bank.

In several cases, notably Dikes 2 to 5, which were built first, considerable settlement occurred after the completion of the lower mud-cell raft, caused, no doubt, by the water which was deflected by the dike cutting under its end, or, perhaps, also by the weight of the dike forcing it down into a bed of quicksand. The number of stories of

mud-cell rafts on the several dikes is given in Table 1, which shows also the order in which the dikes were started.

TABLE 1.

Dike No.	Number of stories.	Dike No.	Number of stories.	Dike No.	Number of stories.	Dike No.	Number of stories.
2	5	1	2	*C*	2	*E*	1
3	5	*A*	2	6	2	7	2
4	4	*B*	3	*D*	2	8	2
5	3						

Three dikes were in process of construction at a time, and work was continued on each until two stories of mud-cell rafts were completed, when the dike was laid by. Later, as required, in order to keep the top of the dike at the proper height, additional stories were put on. After the December flood receded, there were, fortunately, no dangerous rises until several of the dikes had been laid by, but several rises of from 5 to 10 ft. occurred before the completion of any of the dikes. This was of advantage, in that it tended to fill, at least, the lower portions of the dikes and to collect enough deposit around them to increase their stability and to add materially to their resistance to more serious floods. Work was begun in the middle of December, 1908, and the dikes were practically completed by the middle of April, 1909, but a small force was kept on the work until July, in order to have an organization which could be readily enlarged if made necessary by the action of the spring rise.

The water in the river rose to an elevation of 21½ ft. above low-water mark during the latter part of May, and remained high for two weeks. At ordinary low stage, there are from 16 to 18 ft. of water in the deepest part of the channel. The dikes, from Dike 2 up stream, were submerged. The south bank, from Dike 3 down stream, was above high water, but Dikes 3 to 6, inclusive, were under water, excepting 5 or 10 ft. at the shore ends. Dikes 7 and 8 were out of water for practically their entire lengths. The action of the dikes, as a protection to the bank, was reassuring, for none of the bank was lost, except at two places, and there the loss was inconsiderable.

Just below Dike 3, a slice of the bank, 6 ft. wide and 50 ft. long, caved off the day before the crest of the flood was reached, after which no trouble was experienced. Between points 50 and 110 ft. below the

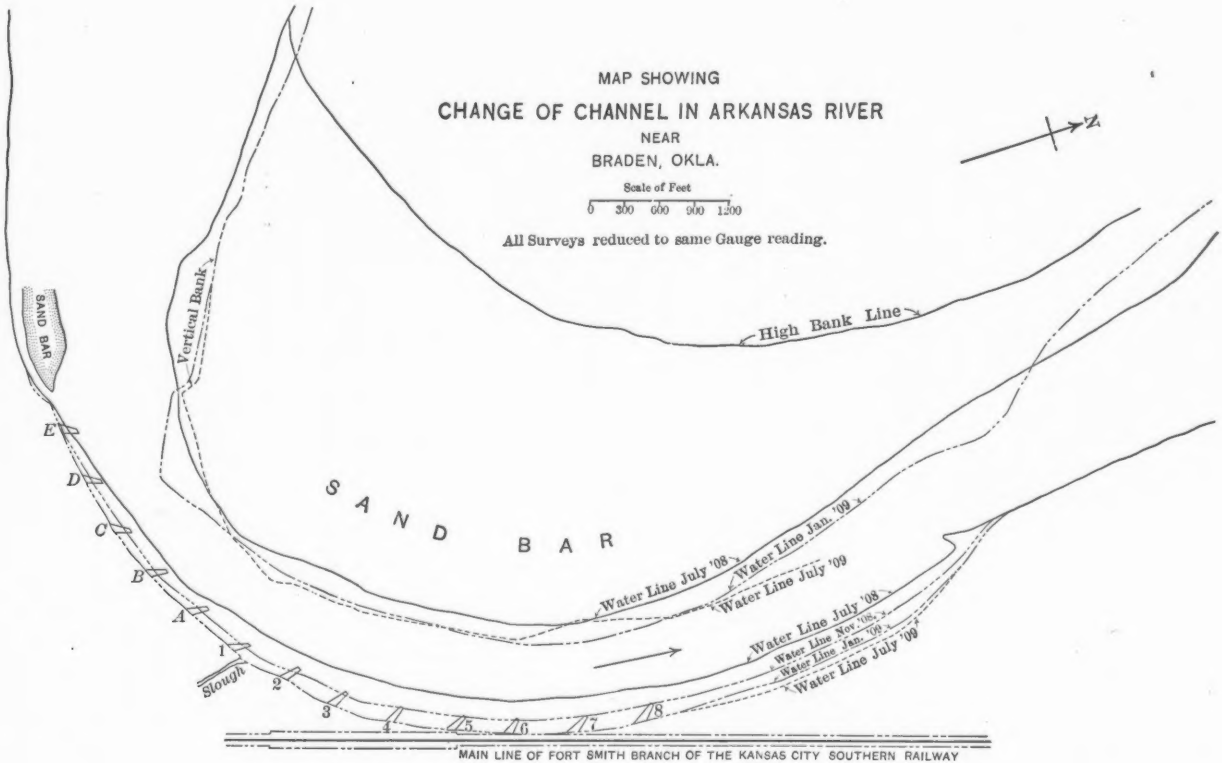

MAP SHOWING
CHANGE OF CHANNEL IN ARKANSAS RIVER
NEAR
BRADEN, OKLA.
Scale of Feet
0 300 600 900 1200
All Surveys reduced to same Gauge reading.
N
SAND BAR
Vertical Bank
High Bank Line
SAND BAR
E
D
C
B
A
1
Slough
2
3
4
5
6
7
8
Water Line July '08
Water Line Jan. '09
Water Line July '09
Water Line July '08
Water Line Nov. '08
Water Line Jan. '09
Water Line July '09
MAIN LINE OF FORT SMITH BRANCH OF THE KANSAS CITY SOUTHERN RAILWAY

FIG. 1.

down-stream side of Dike 5, the bank began falling off on the day the water reached its highest stage. A total width of only 8 or 10 ft. was lost, but several solid fascines were launched before the caving stopped. While the river was rising, a strong current was deflected around the end of each dike, so that it struck the outer half of the next dike down stream. A weaker current turned up stream from the dike and flowed backward, usually close to the shore, until it struck the next dike up stream, from which it was deflected and flowed down stream—until its energy was spent—along a path about midway between the shore and the line along which the current had previously gone down stream. At the flood stage the same action took place in a general way. Sometimes the back current flowed up stream only half way to the next dike before it was deflected down stream by a direct current, and more violent eddies were produced where the strong down-stream currents, which passed over and around the end of a dike, collided with the quieter water below the dike and nearer the shore. The eddies which were produced below Dikes 3 and 5, were so close to the shore—within 30 or 40 ft.—that their energy was not spent before they impinged on the slope of the bank with sufficient force to bore holes in it. It is the writer's opinion that damage of this kind by future floods could be prevented by raising the half of the dike next to the shore, and this has since been done on Dikes 3, 4, and 5.

Since the water has returned to low stage, sand-bars have appeared along the shore between the dikes. These bars have a width (transverse to the stream) of from 20 to 40 ft., and elevations of from 2 to 5 ft. above low-water mark. The cells in the dikes are filled to the same elevation, the deposit extending 5 or 6 ft. outside the dike, except along the outer end and along the outer half of the dike on the up-stream side. In nearly every case the deepest water in the river is just beyond the outer end of the dike.

There is no evidence that any cutting has taken place on the north side of the river, opposite the dikes. Erosion continued on the north bank of the upper end of the large bar shown on the map, Fig. 1. It seems, therefore, that the dikes must take care of the south bank until this erosion progresses far enough down stream to wear off the bar. The south bank, east of a point about 400 ft. below Dike 8, was cut off as much as 75 ft. in places.

The principal damage sustained by the dikes was to their looks. The

outer ends of the upper rafts of Dikes *D* and 3 were washed out of line, so that they hung over the down-stream sides of the next lower rafts, but the upper rafts were not broken apart; and the outer ends of Dikes 2 and 3 were twisted out of shape, but were not detached. Nearly all the dikes settled a little at their outer ends, but none of this damage is sufficient to impair their efficiency. The portion of the upper raft of Dike 5, between 70 and 100 ft. from the shore connection, was torn loose and washed away, and will have to be replaced. This is the only part that was lost. The ability of the dikes to withstand the force of a flood has been established beyond reasonable doubt.

Willows, up to 2 in. in diameter at the butts, are most desirable for constructing the dikes, but, on account of the lack of this kind of material within easy reach of the work, it was necessary to use also pine and hardwood brush. Willows were cut and tied in bundles, 10 or 12 in. in diameter, at Kansas City, and shipped to the site of the work. This supply was limited and, before the work was completed, was shut off by water in the Missouri River bottoms. The pine and hardwood brush could not be tied in bundles and was more expensive and inconvenient to handle than willows, besides causing a less satisfactory grade of work. A total of 18 000 cords of brush and poles (loose measurement, except in willows) was used in the permanent work, and the average cost was $1.55 per cord on board cars at the site of the work. Less than one-sixth of the total amount of brush was willows. No charges were assessed for transportation of material over the Company's line.

The cost of constructing the dikes amounts to $8.15 per ft. of bank protected, counting from the up-stream connection of Dike *E* to a point 400 ft. below the down-stream connection of Dike 8. The economy of this system of work will depend largely on the cost of maintaining the dikes until such time as they will be no longer required for the protection of the bank, which cost is as yet unknown. The portions which are below water will last a long time, as far as decay is concerned, but the portions which are above the sediment collected by the dikes will have a much shorter life. Data as to the durability of the exposed portions of the dikes will be awaited with interest.

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PAPERS AND DISCUSSIONS

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WATER SUPPLY FOR THE LOCK CANAL AT PANAMA.

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On March 16th, 1909, the Chamber of Commerce of the State of New York appointed a special committee to

"* * * make a study, from the standpoint of the requirements of the world's shipping, of the problems involved in the construction of the Canal at Panama, and report what, in its opinion, should be done, engineering and financial considerations permitting, to make the Canal best subserve the needs, for all time to come, of a constantly expanding commerce."

The writer, as a member of that committee, has lately had the opportunity to study many of the very interesting problems brought before it.

Among these, perhaps the question of "water supply for the locks" is one of the most important, on account of the effect it may have on the ultimate capacity of the Canal, and on its ability to meet the future requirements of the service for which it is being built.

Originally, when the proposed locks were of considerably smaller dimensions and the requirements of navigation had, perhaps, not been estimated to their full magnitude, the available water at the Isthmus was considered sufficient for all the needs of the proposed waterway;

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

but now, with the greatly enlarged locks under construction and the increased sizes of the vessels afloat, it seems that this question could and ought to be studied again under these new conditions and with due regard to the ever increasing requirements of navigation.

It is under such impression that the writer ventures to submit to the American Society of Civil Engineers the following considerations, not with the object of criticizing the plans now in execution, but only with the earnest desire of starting among its members a discussion of a subject believed to be of great importance to the country.

To make this analysis clearer, it is well to establish here some of the elements that enter into the problem, viz.:

The main features of the lock-canal system;
The requirements of navigation;
The available supply of water; and
The consumption of water.

Main Features of the Lock-Canal System.

It is well known that the adopted plans for the Lock Canal comprise a high-level waterway between Colon and Panama, consisting of:

a.—One sea-level section, on the Atlantic side, from Colon to Gatun, about 7 miles long.

b.—One set of twin, three-flight locks, at Gatun, with a total lift of 88 ft., connecting the sea-level section with the high-level lake at Gatun.

c.—An artificial lake to be formed at a level of 87 ft. above the sea, to be used for the storage of water for the locks. In this reservoir, to be named "Gatun Lake," will be dug a channel 45 ft. deep and 24 miles long, with submerged banks, for the navigation between the Gatun locks and the Culebra Cut. This reservoir will have an area of 170 sq. miles at high water and about 100 sq. miles at 60 ft. elevation. More than 90 islands will be formed by the flooding of that lake through the hills of the Canal Zone.

d.—The Culebra section, or high-level canal through the Culebra Cut, extending from Gamboa on the lake to the Pedro Miguel locks, about 7 miles, toward the Pacific side.

e.—One set of twin-locks at Pedro Miguel, with a lift of 32 ft. at high water.

f.—One section of intermediate canal, about 1⅓ miles long, between Pedro Miguel and Miraflores, at Elevation + 55 ft.

g.—A set of twin two-flight locks at Miraflores on the Pacific slope, with a maximum lift of 66 ft.

h.—A sea-level section from Miraflores to deep water on the Pacific, near Panama; about 8½ miles long.

The total length of the Canal is nearly 50 miles, of which about 15½ miles are a sea-level canal at tide water, and about 31 miles are at an elevation of 87 ft. above the sea. The remainder is in locks and the intermediate section.

This waterway will have 23 changes of direction, and is to be navigated by a "polygonal route," so as to avoid the various islands and shoals which will be formed in the lake by the existing hills and by the high grounds on each side of the route.

The twelve locks are to be operated with the water accumulated and stored in that artificial lake, from the annual discharge of all the streams emptying into it; that discharge and rainfall being the only sources of water supply for the whole system.

It is expected that the depth of water in the lake, under these plans, will vary only from 47 ft. at high water to 40 ft. at low water, at the end of the dry season.

The purpose of this monograph is to ascertain whether this 40-ft. draft can be maintained at all times in the Canal under the conditions which are possible and probable in that locality.

Requirements of Navigation.

Draft.—The law authorizing the construction of the Canal at Panama, stipulates that:

"Such canal shall be of sufficient capacity and depth as shall afford convenient passage for vessels of the largest tonnage and greatest draft now in use, and such as may be reasonably anticipated." * * *

The first part of these specifications makes it very simple to fix the necessary draft in the canal, as far as the largest vessels now afloat are concerned. With such craft as the *Mauretania, Lusitania, Titanic, Olympic,* etc., requiring more than 38 ft., this figure must necessarily be taken as the actual salt-water draft for which the Canal shall afford convenient passage.

As for the draft that "may reasonably be anticipated" for the future, it is a debatable question which would carry us beyond the limits of this paper, and no one can decide it now with absolute

authority; but a study of the water requirements of an international waterway having permanent locks, would not be complete, from the point of view of true engineering, if due consideration were not given to this most important part of the problem, viz., the probable dimensions of future vessels.

Probable Dimensions of Future Vessels.—Taking as a basis of comparison only the period since steam navigation became fully established, it will be found that in 56 years the following increases in the dimensions of steam vessels have taken place:

Their beam	120%
Their draft	132%
Their length	203%
Their displacement	1 607%
Their driving power	3 082%

A more detailed analysis of the ratio of increase of marine dimensions in the last 25 years only, will show that, if the same gradual average increase be kept for 30 years more, the craft afloat at that time will have dimensions beyond the capacity of even the immense locks now being built at Panama.

With such precedents and a further consideration of the enormously increased facilities for marine construction now at our command, the reduction in the cost of materials, the ever-diminishing ratio of weight per unit of power, the introduction of new metals, new fuels, and new methods for their utilization, added to the well-proven necessity of large units for economy in results, it is permissible to admit that the wonderful gradual development in marine architecture, so marked in these last few years, will not stop just now, when all commercial, industrial, and physical conditions point to the contrary.

For such reasons, the writer believes that, in the not very distant future, commerce will demand, and obtain, for our long-distance trade with the Orient and the South Pacific shores, large freight carriers of low superstructure, wide beam and extreme draft. Such vessels will use the Panama Canal, and this waterway, whether sea-level or lock canal, should be able to meet their requirements when the time comes.

The writer, however, will not presume to predetermine here, even approximately, what the dimensions of such future vessels will be. For the purposes of this paper he will only assume their draft to be at

least equal to that of the largest vessels now on the ways, say 39 ft. 6 in. in salt water, or 40 ft. 8 in. in Lake Gatun.

Clearance for Safe Navigation.—In considering the depth of water required for the convenient navigation of a vessel having such draft, the following items must be borne in mind:

a.—An ocean vessel will draw from 2½ to 3% more on the warm fresh-water lake of the lock canal than on the seas. For this, an allowance of from 1 to 1¼ ft. must be added to the salt-water draft of a vessel drawing 39 ft.

Congress has stipulated 1½ ft. as the extra requirement for Gatun Lake.

b.—In well-protected harbors and docks, where the effects of winds and waves are not felt, a vessel can be made to enter a dock with scarcely 1 ft. of water under her keel; but this demands the greatest of care and the slowest possible speed, and very few navigators would like to assume such risks many times a day.

c.—For locks in a canal exposed to wind and wave fluctuations, considerably more clearance is absolutely indispensable. To cross from one ocean to the other in the proposed lock canal, a vessel will have to pass over twenty-six solid sills in the locks and over many miles of solid rock bottoms in the remainder of the route. To reduce this multiple risk, the designers of the locks have, very wisely, provided for a net clearance, which, at the time of their design, amounted to more than 6 ft. over the miter sills, and 4 ft. in the Culebra Cut, where no large fluctuations from waves are to be feared.

d.—In a large lake like the one proposed for the Lock-Canal system, measuring more than 30 miles in length and 160 sq. miles in area, northers, gales, and storms may produce at any time waves of considerable amplitude. A steady breeze may "back-up" the waters at one end of the lake and depress them at the other for 1 ft. or more at times. If these conditions should happen to cumulate, the total fluctuation of the lake level may be such as to endanger or interrupt navigation.

e.—Preceding considerations show that to permit safe and convenient transit through the Lock Canal, a net clearance of at least 3 ft. in the canal proper and 6 ft. in the lake is needed under the keels of passing vessels.*

* In the Suez Canal, the clearances are restricted as follows: In the narrow channel or canal prism proper 4.86 ft.; in the "Large Bitter Lake" 6.56 ft.; and the canal is incessantly being deepened to maintain such clearances, as the average draft of vessels increases.

f.—Any reduction of such clearances will increase proportionately the risks and dangers of navigation through the submerged channels in Lake Gatun, particularly on the rocky bottoms and short navigating "legs" above Bohio, on account of the heavy fogs that will frequently prevail there.

Available Supply of Water.

On the Isthmus of Panama the meteorological year can be generally divided into two seasons:

The "wet season," lasting from 7 to 9 months; and
The "dry season," lasting from 3 to 5 months.

On the Atlantic side of the Canal Zone, during the "wet season," the rainfall is always abundant, producing an average annual flow of from 8 000 to 14 000 cu. ft. per sec. for the region under discussion; but, during the 3 to 5 months of "dry weather," the rains in that locality cease almost completely, and then the average discharge of all the streams that can empty into the proposed high-level lake at Gatun, will rarely exceed 1 100 cu. ft. per sec. in dry years.

TABLE 1.—Rainfall in the Canal Zone, for the Year 1908, in Inches.

This table shows an actual dry season of more than 150 days (5 months); but, in the computations, a dry season of only 120 days has been considered.

Months.	At Empire.		At Culebra.		At Chagres.		At Rio Grande.		Average.
	Wet.	Dry.	Wet.	Dry.	Wet.	Dry.	Wet.	Dry.	Dry.
July, '07	9.85	.....	9.42	.....	9.60	.....	.....	.....	.....
August	11.28	.....	11.81	.....	12.20	.....	.....	.....	.....
September	10.86	.....	11.36	.....	14.71	.....	.....	.....	.....
October	15.44	.....	15.27	.....	13.62	.....	.....	.....	.....
November	10.40	.....	6.91	.....	9.85	.....	.....	.....	.....
December	.....	1.47	.....	2.30	.....	2.26	.....	1.66	.....
January, '08	.....	0.75	.....	0.91	.....	0.20	.....	1.53	0.847
February	.....	0.00	.....	0.01	.....	0.11	.....	0.00	0.030
March	.....	0.41	.....	0.13	.....	0.41	.....	0.03	0.245
April	.....	1.36	.....	1.67	.....	1.81	.....	0.26	1.275
May	12.91	.....	12.63	.....	13.18	.....	.....	.....	.....
June	8.21	.....	8.86	.....	6.55	.....	.....	.....	.....
Totals	78.95	3.99	76.26	5.02	79.71	4.79	.....	3.48	.....
Average	.....	.....	.....	.....	.....	.....	.....	.....	0.599

Table 1, showing the rainfall in that central district of the Canal Zone, has been computed from last year's report of the Isthmian Canal

Commission. It gives at a glance a clear idea of these variations in the only source of water supply in that locality.

Records of observations covering a period of more than 20 years give the average annual supply available for Gatun Lake as 8 200 cu. ft. per sec. in dry years, and a dry-season average of 1 200 cu. ft. per sec. for all the region above Gatun.

In 1903, one of the driest years, the discharge at Bohio was as follows:

In February,	1 024	cu. ft. per sec.
In March,	636	" " " "
In April,	565	" " " "

averaging only 742 cu. ft. per sec., and it has been calculated that the total available water supply for one year in the driest years, for the whole region under discussion may be only about 6 000 cu. ft. per sec. as the average for such a dry year.

For some of these figures on hydraulics the writer is indebted to the remarkably complete work of Brig.-Gen. Henry L. Abbot, on the "Problems of the Panama Canal."

Storage of Water.—To overcome the difficulty due to this scarcity of water during the dry season, the designers of the Lock-Canal system propose to impound and store in an artificial reservoir, from the surplus of the wet-season discharge, a volume of water sufficient for all the needs of the Canal during the ensuing dry period.

For this purpose a lake is to be created at Gatun, with an area of more than 170 sq. miles, at an elevation of 85 ft. above the sea. A fluctuation of 7 ft. in the level of that lake would give—during the dry season—an available lake storage of 2 ft. above and 5 ft. below the normal working level of the locks to be fed by it.

On a mean area of 160 sq. miles, the volume of water thus stored would be 31 223 708 000 cu. ft.

This quantity, helped by 1 000 cu. ft. per sec.—the average daily contribution of all the tributary streams discharging into the lake during the dry season—will be the only supply from which the Lock Canal is to be operated, and is thought, by its designers, to be sufficient for all its needs during the 100 or more days of water scarcity.

It is perhaps better to quote here the exact words of the engineers who reported on this important matter to President Roosevelt, on February 17th, 1909.

"It is proposed to fill this lake (Gatun) during the rainy season 2 feet above its normal level, and to draw it as needed during the dry season. It is computed, that by drawing it 5 feet below normal level, which draft would leave 40 feet of water through Culebra cut, the supply in a dry year would be sufficient to serve from 30 to 40 lockages up and an equal number of lockages down daily. Each lockage might consist of a single large vessel, or a fleet of smaller vessels capable of being in the lock at one time. * * * The water supply in sight is so much greater than any need that can be reasonably anticipated that the best method of securing more water when the time of need arrives does not require to be considered now."*

The writer, however, applying to this problem the same study that would be required if it was a commercial proposition, believes that, under the conditions existing, or possible, at Panama, the volume of water thus stored in the lake will prove to be insufficient for the demands of the Lock Canal when its service reaches the duty specified by its engineers in the words above quoted.

To ascertain this, within the limits of an approximate, but safe, estimate, the following computations are submitted.

Consumption of Water.

In a system of locks fed by an artificial storage reservoir, under the conditions proposed for the Lock Canal, the supply of water must be sufficient to provide for:

I. Surface evaporation,
II. Border evaporation or "fly-off,"
III. Percolation through the bottom and sides of the reservoir,
IV. Water needed for lockages,
V. Normal waste, contingent to lockages,
VI. Water used for hydro-electric plants, at Gatun and Miraflores,
VII. Losses due to leakage at gates and valves,
VIII. Water wasted by maintenance operations,
IX. Losses due to false maneuvers and accidents,
X. Losses of water at spillways and regulating gates,
XI. Miscellaneous losses, local consumption, etc.

I.—Surface Evaporation.—It is well known that surface evaporation from water exposed in the open takes place at all times; but those who have had to economize water in regions where this industrial

* Report of Board of Engineers, Feb. 17th, 1909, Document 1458, p. 10.

element is scarce, will remember what a persistent consumer evaporation is, and how difficult it is to measure accurately its activities on a large reservoir.

The "evaporating pans," "atmometers," "evaporimeters," and other instruments of small size that are generally used to gauge daily evaporation are, to say the least, extremely inaccurate, in practice. Any number of them distributed at various stations in a territory covering many square miles will not show, on a given day, the same reading; and their average will not give the actual evaporation on a lake that would cover that same territory. This is due to the different way in which the wind, temperature, and other meteorological elements will affect the convex, rippled surface of a large lake, or the small, confined area of an instrument. On bright, windy days the difference may amount to 25 or 30% in favor of the larger surface because the action of the wind affects the results more than the other causes.

In the tropics, under conditions quite similar to those at Panama, the writer has found surface evaporation during the "dry season" to vary from 0.22 to 0.44 in., according to the month, the temperature of the air and water, the altitude, the barometric conditions, the degree of humidity, the prevalence of winds and fogs, the extension of the surface, and the surrounding locality, it being at the highest either in February or March. In these months, on certain warm days when the moisture in the air is very high, evaporation is much reduced during the hours of calm; but it increases rapidly as soon as the tropical breezes start, reaching sometimes as much as 0.75 in. on very windy, clear days.

In the Gatun district, on account of the exposed condition of the "Zone," the air currents are strong and quite constant, with breezes frequently attaining a velocity of from 8 to 10 and even 32 miles per hour.

Under such conditions, a large lake, with more than 170 sq. miles of warm, shallow waters, swept by such breezes, and exposed to the heat of a tropical sun (125° to 130° in the sun) will certainly have a very high rate of evaporation. And when it is considered that the ripples formed by the wind on the surface of such a vast sheet of water will increase immensely its effective evaporative area, it will be seen that the actual loss on the lake can be considerably more than what would be shown by instruments of small area.

It must be remembered, however, that in the tropics, the large amount of water evaporated from a lake by the combined action of the sun and wind, will produce at times, in the hours of calm, dense fogs during which very little evaporation will take place. Though such fogs, of course, will be more frequent in the rainy season, it is well to anticipate at least a total of 200 or 300 hours of them during the strictly dry months. This will decrease the total evaporation by about 8 or 10%; not enough, however, to counteract the influence of the wind, leaving average daily loss still above what experimental instruments of small diameter may show, at various stations.

For these reasons the writer believes that the actual daily loss of water by surface evaporation, if it could be measured accurately on the lake, would be found to be much larger than what has been assumed as an average for Lake Gatun, and nearer the following figures:

For December	0.10 to 0.15 in.
January	0.25 "
February	0.29 "
March	0.30 "
April	0.24 "
May	0.10 to 0.12 "

The average is from 0.24 to 0.246 for the five months of dry weather.

When comparing these figures with others, it is well to remember that in Tables 4 and 6 the actual rainfall on the lake is deducted for each month; and will reduce correspondingly the above apparently high figures.

The writer believes that the system of calculating the evaporation on a reservoir of varying surface, by multiplying its area by the average daily evaporation of the whole year, may produce errors which may be of importance in some localities, when only the dry season is being studied.

II.—Border Evaporation.—The lowering of Lake Gatun's level from high to low water, in every dry season, will uncover an area of land measuring more than 100 sq. miles.

Under ordinary conditions, if these "border lands" remain covered with the vegetation which usually grows on river sides in the tropics, and are kept moist, as they will be, by capillarity and by the constant

fluctuations of the water's edge, due to the wind, waves, and other causes; this "fly-off," as it is termed in the West, from these moist borders may reach a considerable amount per day on the 425 miles of shore of that lake, at the temperature which the thermometer will register there in the sun.

After due allowance for drainage, run-off, etc., the water evaporated by this daily uncovering of shores and low lands, may be estimated at 30 cu. ft. per sec.

This, distributed on the lake, will be:

For the	1st	month,	160	sq. miles	0.0066 in.
" "	2d	"	140	" "	0.0076 "
" "	3d	"	120	" "	0.0088 "
" "	4th	"	100	" "	0.0104 "

III.—Percolation.—When a deep storage reservoir is formed on natural ground, there is usually quite a loss of water by percolation through its bottom and sides.

This "seepage" depends principally on the porosity of the soil; the number of seams and fissures (if any); the depth of the water; its temperature; the surrounding locality; the geological conditions, etc. It may vary, in ordinary canals and ponds, from 1 to 4 or more inches per day, measured as a lowering of the water level.

In a large reservoir like the one proposed at Gatun, with more than 200 sq. miles of wetted area, in a region showing varied geological formations and a basin of natural soil, covered in part with tropical vegetation, it is more than prudent to admit that a measurable percolation is possible and quite probable.

Unless the whole 200 sq. miles of that valley be stripped and examined, this probable percolation cannot be estimated in advance, even approximately. Unless every square foot of that vast area is made impervious by artificial means, the loss of water by seepage cannot be authoritatively dismissed as "an impossibility." It can only be guessed at now, and will be ascertained only after the lake is filled. Meanwhile, the question of the possibility of an appreciable amount of percolation through that immense lake bed will remain as one of the most serious risks in a matter upon which depends the ultimate cost and perhaps the success of the whole Lock-Canal proposition.

Under the circumstances, the writer believes that a margin of safety

ought to be assumed against such risks, and that it should be duly proportionate to the importance of the results at issue.

In the absence of experimental data especially applicable to the whole lake basin, an assumption could be based on the following tentative elements:

a.—Infiltration area of the lake bed over 200 sq. miles at high water;

b.—Depth of water, 85 ft. at high water and shallower at the contours;

c.—Top soil covering naturally thicker and more impervious in the lower parts of the bed than in the higher regions above Bohio and at the sides of the basin;

d.—Temperature of the stagnant waters in the lake above 90° Fahr. in the shallow parts and about 82° Fahr. in the deeper bottoms;

e.—That the foregoing elements are about compensating, leaving the loss of water nearly proportionate to the head, and depending on unknown subsoil conditions;

f.—That joints, seams, and fissures are generally abundant in sandstones, conglomerates, and other rocks like those found in the Canal Zone.

g.—That the absence of permanent lakes and springs in the whole region, strongly indicates either good drainage or ample underground communication with water-absorbing substrata.

Under such assumptions, the writer would consider the empiric rule, $H \times 0.01$, or 0.01 in. per foot of head, a prudent one on which to base a preliminary computation. This, owing to the ever decreasing head of the water in the lake, would give, during a dry season, the following probable depressions:

For the	1st	month	0.4750 in.
" "	2d	"	0.4450 "
" "	3d	"	0.4150 "
" "	4th	"	0.3770 "

To many, these figures will appear high, and it is hoped that they will prove to be too high; but, without accurate data to the contrary, there is no competent authority to declare them so in advance. It is altogether a question of judgment as to the amount of risk to be taken.

To prove, however, that these figures are based on elements that

deserve some consideration, the following data, published in *The Canal Record* for 1908 and in the Isthmian Canal Reports, may be interesting if not convincing:

"Leaks in the Camacho Reservoir (near Empire in the Canal Zone).

"Leaks in Camacho Reservoir near Empire have made it necessary to draw the water from Elevation 358 to 340 (18 ft.) in order that the fissures in the trap rock of the side hills may be filled with clay. It has been ascertained that the leaks are through the side hills and that the dam is in no way affected."*

TABLE 2.

SEEPAGE FROM SOME WELL-KNOWN CANALS.†

	Loss of level, per day.
Chesapeake Canal	5.04 in.
Chenango Canal	5.00 "
Erie Canal	5.00 "
Morris Canal	8.50 "
Delaware and Raritan Canal	8.50 "
Canal du Lanquedoc	2.76 "
Canal "Naviglio Ide"	10.80 "
Canal "Martesana"	18.00 "
Hoover ditch, Colorado	14.40 "
Kings River and Fresno	20.04 "
Fort Morgan, Colorado	60.00 "

From the report of the very able engineer in charge of the seepage experiments for the foundations of the dam at Gatun, the following data are also obtained:

"Some of the rocks (in the Gatun Valley) are of open texture and permit the flow of water" (page 127).

"The principal water-bearing medium seems to be the joints and seams which cut through the various strata and allow the passage of large quantities of water" (page 127).

"The material has well-defined joint planes and seams through which water passes with velocity enough to throw the stream from 1 ft. to 1½ ft. into the opening" (page 165).

The same authority, as a result of most interesting and carefully made experiments, gives the seepage flow through the Gatun soils experimented upon, and these are collated in Table 3.

* *The Canal Record*, Vol. I, p. 402.

† Report, Isthmian Canal Commission, 1908, p. 196.

TABLE 3.—SEEPAGE VELOCITY IN GATUN SOILS.

	Feet per second.	Inches per day.
For clay on 10 ft. head	0.000028	29
For soft sandy rock	0.000045	46
For clay and sand	0.000083	86
Averaging for the three	0.000052	54
For clay and sand, 8½ ft. head	0.000070	73
For clay and sand, 85 ft. head	0.000705	731
(The greatest velocity obtained with 34 tests, under 10 ft. head)	0.000450	466
Blue sandy clay, 115.3 head	0.000193	200
Blue sandy clay, calculated on 84.5 ft head	0.000014	14
Stiff blue clay, calculated on 84.5 ft. head	0.000002	2
Average of these two on 84.5 ft. head	0.000008	8

The figures* in Table 3 prove that all the overlying soils and materials experimented upon are sufficiently permeable to allow more than usual percolation.

With such materials, the quantity of water which can disappear through the bottom and sides of that lake will evidently depend on the underlying strata and substrata, their thickness, hydraulic gradients, the presence of fissures, etc., etc. Many of these elements are now unknown, and many are still under investigation; but the testimony taken on this matter—in connection with the foundations for the locks—tends to prove the existence in that locality of substrata through which there is a slow but regular flow of water toward the sea.

In a bed of rock or conglomerate formation that geologists claim has been disturbed three times, there might be more than joints and seams, perhaps fissures and crevices, through which the impounded lake waters may get egress and slowly flow toward the ocean, as they did in the Camacho Reservoir.

In ordinary cases, when dealing with a reservoir having an ample "replenishing supply" to draw from, this matter would not be so important; but in Gatun Lake, all the inflows during the dry season, amounting to only 0.25 in. per day, are scarcely enough to keep up with the evaporation loss alone. Any additional loss becomes a direct draft from the stored supply and is irremediable until the next rainy season.

IV.—Water Needed for Lockages.—The conditions under which the Lock Canal is to be operated, according to the proposed plans, are:

* Report, Isthmian Canal Commission, 1908, p. 134.

The locks are to be 110 ft. wide, with the length necessary to admit vessels 1 000 ft. long.

Those at Gatun are to be in flights of three, with an average lift of 28⅓ ft.; those at Pedro Miguel are to have a lift of from 30 to 32 ft.; and those at Miraflores are to be in a flight of two, with lifts of 30 and 40 ft.

These locks will be in duplicate, making twelve chambers in all. Those on one side are to be used for vessels going in one direction; those on the other side for vessels going in the other, or by "double-track navigation."

All craft are to be moved in and out of the locks by special appliances, and will not be permitted to use their own motive power.

No restrictions which would cause delay to boats, such as parking, classifying, etc., will be enforced for the regulation of traffic through the locks.

Safety appliances will be used at every lockage, both for entering and leaving.

It is understood that no other sources of water supply than those now planned, are to be used, and that the two-flight locks at Miraflores will have no other source of water supply, during the dry season, than the tailings from the Pedro Miguel locks on the lake above.

If additional reservoirs at Alhajuela or other points in the cachement basin are established later, or pumping stations are installed to refill the lake from the sea-level sections, they will modify the conditions of the problem. Not having been included in the proposed plans and estimates, these improvements are not considered in the writer's calculations.

Theoretical "Lockage Water" Consumption.—The exact volume of water needed for one lockage, under any given condition, can be computed with accuracy; but, in practice, it will vary considerably with the operating circumstances, the working level of the lake, the tides, etc.

The theoretical lockage volume for the upper three-flight locks at Lake Gatun at the 85-ft. level is as follows:

Gatun upper lock.....	28.33 ft. lift....	3 614 908 cu. ft.
Pedro Miguel.....	30 " "	3 828 000 " "
Total volume at normal 85 ft.....		7 442 908 cu. ft.

These figures do not include any "extras" for the normal waste incidental to the operation of locks, irregularity of service, leakage at gates, valves, etc., nor any correcting coefficient for the larger consumption of the "three-flight" locks.

If this coefficient be applied for the Gatun locks, the volume for one single passage through the lake, in one direction, would be:

For Gatun locks	4 518 635 cu. ft.
For Pedro Miguel	3 828 000 " "
	8 346 635 cu. ft.

If the intermediate 600-ft. chambers could be used at Pedro Miguel at every lockage, the volume for one lockage would be:

For Gatun Locks, as above	4 518 635 cu. ft.
For Pedro Miguel Lock	2 508 000 " "
For one lockage	7 026 635 cu. ft.

But the 2 508 000 cu. ft. then used by the Pedro Miguel Locks will not yield "tailings" enough for the 3 828 000 cu. ft. needed by the "two-flight locks" at Miraflores when these are used full, nor even for the 3 062 400 cu. ft. required if they could also be operated with the intermediate gates for part of the time.

Assuming this, however, to be obtainable sometimes, the lowest consumption that could then be shown would be:

For Gatun Locks, as above	4 518 635 cu. ft.
For Pedro Miguel (using enough water to feed the Miraflores Locks, with these working 75% at intermediate chambers)	3 062 400 " "
Minimum volume for one trip	7 581 035 cu. ft.

As it may be claimed, however, that great economy of water can be obtained by reducing the coefficient of the "three-flight locks" and by using the Miraflores locks at intermediate gates for 75% of their lockages, particularly during the first years of operations, it is well to know what would be the results under such exceptionally propitious conditions:

Assuming, then, that favorable circumstances would permit the reduction of the lockage coefficient of the Gatun locks to, say, only 1.08, the figure adopted by Gen. Abbot, the lockage water will measure:

For Gatun Locks.....say	3 904 100 cu. ft.
For Pedro Miguel Locks, using the volume necessary to operate the Miraflores Locks at 75%.....	3 062 400 " "
Total for one lockage going to Panama..	6 966 500 cu. ft.

The use of the intermediate gates, however, will be objectionable in daily practice, on account of the great number of safety appliances to be used for each craft at each lock, and the necessity of moving all vessels at extremely slow speed by electric capstans, cables, etc., instead of by their own power. Owing to the diversity of operating conditions in these series of locks, it is probable that better service will be obtained if full lockages are taken with several vessels in fleet at one time, instead of using the intermediate gates for each craft.

In considering the capacity of the Canal, the average number of lockages assumed is 20 trips in one direction through one set of locks and 20 trips in the other through the other set, with a consumption of 333 865 400 cu. ft. in 24 hours. It is assumed that any reduction of service in one day will be made up on other days.

The drawing of that amount of water from a lake having, say, 160 sq. miles of surface (the mean area for the first month), will produce a lowering of its level amounting to 0.900 in. per day.

As this daily volume is constant, and the net area of the lake will diminish every day, owing to the lowering of its level, and by the uncovering of islands, slopes, etc., the depth of water lowered every day—or the daily depression of the surface—will increase inversely, giving the following average daily depressions:

For	1st	month,	mean	area	of	lake	160	sq. miles....	0.900 in.
"	2d	"	"	"	"	140	"	....	1.0285 "
"	3d	"	"	"	"	120	"	....	1.200 "
"	4th	"	"	"	"	100	"	....	1.440 "

These are the theoretical depressions for "lockage water" only. In practice, they will be exceeded considerably. They may also vary

somewhat with changes in the mean areas of the lake, taken here in round numbers, as shown.

V.—Normal Waste of Water During Lockages.—With the three different kinds of locks that will be used in the Lock Canal, viz., the three-flight locks of from 28½ to 31-ft. lift at one entrance of the lake; a 30 to 32-ft. single-lift lock at the other, followed by a 35-ft. two-flight lock depending on it for its water supply, it is easy to imagine more than twenty different combinations of conditions which can affect the quantity of water consumed by the passing of a certain number of vessels, as the following few examples will show:

Let L = the net contents of the lock chamber, in cubic feet,
and B = the displacement of a vessel in the lock, in cubic feet.

Then, if, at the Gatun Locks, for an ascending vessel, the lock is found empty, the lockage will cost $3\ L + B$ cubic feet; if the lock is found full, the volume of water used will be only $L + B$ cubic feet, showing a possible variation from single to double.

These Gatun Locks (being in a flight of three, without lateral reservoirs), will not permit advantage being taken of any occasional favorable operating conditions; and, in passing a vessel through them, circumstances may even combine to compel an extra waste of water which could be avoided if they were "single locks."

At the one-flight lock of Pedro Miguel, the water used by vessels passing in train through one lock and returning by the other, will be considerably more than would otherwise be needed, if said vessels could pass ascending and descending, alternately.

In other waterways, great economy of water could be obtained in single locks like those at Pedro Miguel, by the use of their intermediate locks and also when the level of the lake lowers; but in the Lock Canal, as designed, if such economies were attempted at Pedro Miguel, the "tailings" from its locks would not be sufficient for the supply of the two-flight locks at Miraflores; additional water to complete the consumption of these would have to be furnished, either from other sources, or direct from Lake Gatun.

This explains why no reductions are made for the lowering of the lake, nor for the use of intermediate gates at Pedro Miguel, in the preceding computations.

The foregoing few examples show how important the "normal

waste" can be, and how it depends on varying conditions against which provision must be made. It is true that, by strict regulations, measures may be taken to reduce this waste somewhat, but such a course will produce annoying delays and will be a serious obstacle to free and rapid navigation. In a commercial canal, open to the world's traffic, it will be impracticable to regulate transit so as to secure always the best conditions for economy of water, and some will have to be sacrificed in order to save time or obtain the number of lockages desired.

Under the conditions above analyzed, and probable in the Canal Zone, the efficiency of the system may vary the waste from 10 to 25 per cent.

Supposing that by good management and perfect regulation of the passage of vessels, the "lockage waste" is reduced to only 6.66%, the average daily depression of the lake surface, by this normal waste would be:

For the	1st	month,	0.0856 in.
"	2d	"	0.0978 "
"	3d	"	0.1141 "
"	4th	"	0.1370 "

It might be argued that, by having a smaller number of lockages a day, much water could be saved; but this will reduce accordingly the total capacity of the Canal, bringing it below the already too small number of "20 lockages a day," here assumed.

The writer believes that a great tonnage will pass through the Isthmus, not only from other nations but from our own inter-coastal commerce with our Pacific shores and with the Orient. He believes that the Lock Canal in the very immediate future will be insufficient for the demands that will be made upon it; that maximum full lockages with as many vessels crowded in the locks as safety will permit, are the conditions for which to look forward.

If the necessity of saving water does not permit the use of the locks whenever and however needed, without imposing annoying delays and regulations, then the system is insufficient and *per se* inadequate for the requirements of our commerce, of the navy, and of the world at large.

VI.—Water Used for Hydro-Electric Plants.—The power needed to operate the twelve locks; to move vessels "in and out" of locks by

electric capstans; to control mechanically the many appliances composing said plants; and to light the canal, will vary considerably, according to the designs adopted, many of which are still under study.

It is intended to use about 6 000 h.p.; part of it to be developed at Gatun and part at Miraflores. It is probable, however, that lack of water will suggest serious alterations in these plans. Mature consideration of this question will prove the necessity of either reducing the hydro-electric plant and locating it all at Gatun or using steam power, unless a dam is built at Alhajuela or other places to store more water and incidentally produce the power.

Assuming that 3 200 h.p. are to be developed at Gatun, the volume of water consumed under an average head of 72 ft. will be 533 cu. ft. per sec.

The lack of water at Miraflores and the irregularities of head due to tides, etc., will make the production of electricity at that spot very uneconomical, as the water will have to be furnished from the lake, and the head available will average only 50 ft. The production of 2 800 h.p., under such conditions, will imply a consumption of 667 cu. ft. per sec., bringing the total water used by the hydro-electric plants, for net power only, to 1 200 cu. ft. per sec.; this is without including the usual allowances for waste.

In Tables 4 and 6 the two plants are shown separately, with a total consumption of 1 200 cu. ft. per sec.

The resulting lake depressions for each pumping station will be:

For the	1st	month,	0.1392 in.
"	2d	"	0.1591 "
"	3d	"	0.1856 "
"	4th	"	0.2227 "

With a dam at Alhajuela, this item would disappear from the Lake Gatun budget, while additional storage would be secured, without increasing the high-water level on the lake, and the corresponding "operating oscillations," which are already too great for safe and convenient handling of craft in the locks.

VII.—Losses Due to Leakage at Gates and Valves.—The gates and valves in the proposed locks will have dimensions never before attained in lock construction.

The steel gates will have leaves more than 80 ft. high and 65 ft.

wide, weighing perhaps 500 tons each. The locks on Lake Gatun will have 44 of such leaves, and more than 80 operating valves exceeding perhaps 10 ft. in diameter, with their corresponding seats and joints under pressure.

It will be almost impossible, in practice, to keep such ponderous appliances absolutely tight, and a considerable "normal waste" must be expected and allowed for. Even the most improved appliances, if fitted too tightly, could not be operated with the speed that lock service requires. The lock gates when new, must be built with a clearance of at least 1½ in. in order to allow for future wear; and it may be said that a perfect condition of tightness in them could never be maintained long, if it is ever attained.

This normal leakage may vary from 8 to 12% of the lockage water under the best conditions of practical equipment.

Taking only 8%, or 295 cu. ft. per sec., the daily depressions produced on the lake will be:

For	the 1st	month,	0.0684 in.
"	2d	"	0.0782 "
"	3d	"	0.0912 "
"	4th	"	0.1095 "

VIII.—Water Wasted by Maintenance Operations.—The enormous amount of "metal work" forming the active parts of the locks, which will be almost continually in contact with water, will demand, in the Canal Zone, incessant care, repeated cleaning, scraping, painting, and other special protection against the tropical elements. Much of this work must, necessarily, be done during the dry season only.

With perhaps more than 30 000 tons of movable lock gates; miles of submerged pipes, culverts, joints, cables, chains, safety appliances, etc.; with more than 80 immense "by-pass," "sluice," and "auxiliary" valves; with four enormous rolling gates, bumpers, their operating appliances, and submerged tracks, and with miles of masonry walls, it is prudent to admit the probability of many compulsory stoppages for running repairs and for maintenance. In many cases, it will be possible to employ divers, but in others it will be necessary to empty the locks and sacrifice the large quantities of water they contain.

Admitting that, when possible, all renewals, running repairs, flushings, etc., are left for the wet season, and that only unavoidable losses

will be permitted during the season of scarcity, it seems that an allowance of 65 cu. ft. per sec. would be a conservative figure to cover these contingencies. To this must be added the amounts chargeable to the power plants and hydraulic motors operating on Lake Gatun, making together about 125 cu. ft. per sec. The corresponding depressions on the lake will be:

For the	1st	month,	0.0290 in.
"	2d	"	0.0316 "
"	3d	"	0.0385 "
"	4th	"	0.0460 "

In considering these figures it is well to bear in mind that the locks will be forever in continual use, with no "long dead seasons" during which complete overhauling could be undertaken. It may be claimed that the locks, being in duplicate sets, could be emptied at any time for general overhauling. This is true; but the safety appliances and towing devices now arranged for "double-track" service, will make "single-track operation" very dangerous, and will reduce considerably the tonnage capacity of the canal, when attempted.

The repairs will have to be made when needed, and as often as needed. Hence the apparently large allowance for this contingency.

IX.—Losses Due to False Maneuvers and Accidents.—With the many protective features now under study for these locks, and the many regulations that will have to be enforced on the passing vessels, no serious accidents in the locks are anticipated. The jamming of gates and valves by floating débris; the occasional breaking of a valve spindle or gate trunnion; some mishap at the rolling gates; perhaps some trouble at the water-wheels, are the most probable causes of some extra loss of water, which, with care, can be reduced to a minimum.

Even with the best of management, however, false maneuvers will be a constant source of danger, and will cause great waste of water, particularly when handling foreign craft with crews not familiar with the service, and at night in locks of unprecedented dimensions, where the oscillations of the working level will be at times considerable, and the signaling distances very long.

Accidents, however, can be predicted, but not estimated; that they may happen is now too well proven by the very serious one of June, 1909, in the Sault Ste. Marie Lock, when three vessels were sunk and

the lock put out of commission after losing enormous quantities of water.

To cover only normal contingencies, a margin of 125 cu. ft. per sec. is taken, giving the following lake depressions:

For the	1st	month,	0.0290 in.
"	2d	"	0.0316 "
"	3d	"	0.0385 "
"	4th	"	0.0460 "

That these figures are conservative will perhaps be admitted, if the following data are analyzed:

The delays to navigation in the Poe Lock on account of breakages have been reported as follows:

In a total of 2 007 days of operation in the single Poe Lock at St. Mary Falls, Mich., there have been 1 104 hours of stoppage due to breakages, as follows:

Gate engines	282 hours.
Gate cables	145 "
Sheaves for cables	44 "
Obstructions against miter sills	37 "
Grounding on platforms	26 "
Fouling of tow-lines	26 "
Jamming of vessels	12 "
Gates	204 "
Pressure pipes	48 "
Gratings	6 "
Changes in pressure system	249 "
Miscellaneous	25 "
Total	1 104 hours.

These were only current repairs during operation.

In the Poe Lock, all "necessary" repairs and overhauling of the lock and machinery are always made every winter when the canal is not in service, the foregoing delays being only for "accident" repairs.

If this is the case for only one lock having only 4 gates and a 20-ft. lift, what will be the figures for 12 locks of larger sizes with 30-ft. lifts, 84 gates three times larger, in continual use, and performing a duty seven times larger under conditions extremely more unfavorable?

This leaves out of consideration the additional contingencies due to the spillways, rolling bumpers, auxiliary drawbridge dams, capstans, and towing appliances, not used at all in the Poe Lock.

X.—Losses at Spillways and Regulating Gates.—The regulating appliances at the spillways must be, by construction, extremely strong and of ample size, so as to be able to resist the enormous discharges of "flood times."

The "spillway" at Gatun will be more than 800 ft. long, with 21 steel gates, each 38 ft. wide and with 16 ft. of water depth. In order to be operated easily and quickly, these active parts cannot be made, in practice, to fit too tightly; hence this source of leakage, being continual, may amount to very considerable figures.

For the normal leakage at these extra large regulating gates, it would not be good practice to take less than 200 cu. ft. per sec. for the conditions that will prevail in the Canal Zone; but, allowing for the gradual lowering of the lake level, and to avoid criticism, 125 cu. ft. per sec. is taken as the average loss.

The corresponding deflections will be:

For the 1st month, 0.0290 in.
" 2d " 0.0316 "
" 3d " 0.0395 "
" 4th " 0.0000 "

XI.—Miscellaneous Losses.—As the lake will absorb and control all the streams of the central region of the Canal Zone, for miles around, it must in return furnish water, directly or indirectly for:

A.—All the needs of the populations and industries that may locate around its borders, and on its numerous islands;

B.—The consumption and replenishing of all passing vessels and crafts using the lake; and

C.—All the losses and wastes at the intermediate section from Pedro Miguel to the sea-level section at Miraflores, due to evaporation, percolation, fly-off, etc.

For all these losses 300 cu. ft. per sec. is assumed, giving the following depressions:

For the 1st month, 0.0464 in.
" 2d " 0.0536 "
" 3d " 0.0615 "
" 4th " 0.0742 "

It will be observed that in the foregoing computations, many of the figures are debatable, being matters of judgment. That is why such lengthy and tedious reasonings have been given in order to state the grounds on which these figures are based. If the estimated losses of water are too large, any economy realized on them after the canal is built will be a clear gain from which to draw for future enlargements of the service.

The writer believes that, in questions of such importance, any approximation on the safe side is better than a "poor guess," and, in consequence, he submits his figures only to invite discussion on the debatable points.

The remote possibility that a lake created at an enormous expense and on which depends the whole Lock-Canal system, might not be up to its estimated capacity when finished or later, merely for lack of water, deserves, at least, to be well studied by civil engineers.

Replenishing Water Supply.

The only replenishing supply for Lake Gatun—after the last rain of a wet season has fallen—is, as has been stated, what it will receive from all the tributary streams that empty into it, plus whatever occasional rain may fall on its surface.

Records show that the available supply from these two sources will rarely average more than 1 200 cu. ft. per sec., and that this may be considerably less in "dry years."

In these computations the local precipitation has been considered separately, and the corresponding monthly amounts have been credited to the lake each month, as shown in Tables 4 and 6; the rainfall of 1908 is taken as an example, being the latest available, but by no means the driest.

As the plane of drainage discharge in that Chagres-Gatun Valley will be raised 87 ft. above its original level on an area of 170 sq. miles, the part of the dry season's inflow corresponding to the drainage of a great part of that area will disappear from the lake's budget, thus reducing to that extent the amount of "dry weather" replenishing inflow.

That being the case, if figuring on the driest years, the inflow from all tributaries above the lake could be taken at 1 000 cu. ft. per sec. for the months of scarcity; but the writer has assumed that a remnant

of the rainy season run-off will overlap into the beginning of the dry season and increase the average of its first month's flow to about 1 500 cu. ft. per sec.

This will give a "dry season" average of 1 217 cu. ft. per sec., and add to the lake level as follows:

For the 1st month, 0.3466 in.
" 2d " 0.2118 "
" 3d " 0.2790 "
" 4th " 0.4810 "

In the extra dry years which occur at intervals, the foregoing amounts will be considerably less after deducting the precipitation on the lake, etc.

TABLE 4.—LAKE DEPRESSIONS DURING THE DRY SEASON: 20 ROUND TRIPS.

Losses.	DAILY DEPRESSIONS OF THE LAKE LEVEL, IN INCHES.			
	1st month, 160 sq. m.	2d month, 140 sq. m.	3d month, 120 sq. m.	4th month, 100 sq. m.
I. Surface evaporation	0.2500	0.2900	0.2950	0.2500
II. Border evaporation	0.0066	0.0076	0.0088	0.0104
III. Percolation	0.4750	0.4450	0.4150	0.3770
IV. Lockage water only (20.)	0.9000	1.0285	1.2000	1.4400
V. Normal waste for traffic	0.0856	0.0978	0.1141	0.1370
VI. Hydro-electric plants, Gatun	0.1392	0.1591	0.1856	0.2227
VI. " " " Miraflores	0.1392	0.1591	0.1856	0.2227
VII. Leakage at gates and valves	0.0684	0.0782	0.0912	0.1095
VIII. Water used for maintenance	0.0290	0.0316	0.0385	0.0460
IX. Losses by false maneuvers, etc	0.0290	0.0316	0.0385	0.0460
X. Losses at spillway	0.0786	0.0480	0.0395	0.0000
XI. Miscellaneous losses	0.0696	0.0804	0.0922	0.1112
Total losses per day, in inches	2.2702	2.4569	2.7040	2.9725
Credit daily inflow, average 1 100 cu. ft. per sec.	0.3466	0.2118	0.2790	0.4810
Net depression per day, in inches	1.9236	2.2451	2.4250	2.4915
Loss of draft per month, in inches	57.71	67.35	72.750	74.745
Credit, rainfall on lake (1908)	0.84	0.30	0.245	1.275
Net loss by months, in inches	56.87	67.05	72.50	73.47

Total loss of draft in canal during one Dry Season.

After 30 days	56.87 in.	= 4.74 ft., say	4 ft.
" 60 "	123.92 "	= 10.32 " "	10 "
" 90 "	196.42 "	= 16.36 " "	16 "
" 120 "	269.89 "	= 22.48 " "	22 "
(year 1908) " 150 "	343.36 "	= 28.61 " "	29 "

TABLE 5.—RESULTS OF ONE DRY SEASON'S OPERATIONS (20 TRIPS), STARTING WITH LAKE FULL, UP TO + 87-FT. MARK.

	Depression.	USABLE SALT-WATER DRAFT LEFT IN CANAL (3%).			
		Absolute.	3 ft. clearance.	4 ft. clearance.	6 ft. clearance.
Beginning of dry season....	0.00 ft.	45.59 ft.	42.60 ft.	41 ft.	39 ft.
After 30 days.....	4.74 "	41.00 "	38. "	37 "	35 "
" 60 "	10.32 "	35.59 "	32.60 "	31 "	29 "
" 90 "	16.36 "	30.36 "	27.36 "	26 "	24 "
" 120 "	22.48 "	23.79 "	20.79 "	19 "	17 "
" 150 "	28.61 "	17.84 "	14.84 "	13 "	12 "

The Isthmian Records show many years of more than 150 days dry weather.

Table 5 shows that:

After the first month of operation, the contemplated 41-ft. draft in the Canal will have disappeared.

After 60 days, vessels drawing 33 ft. cannot use the canal with safety.

" 90 " " " " 28 " " " " " " "

" 120 " " " " 21 " " " " " " "

" 150 " " " " 15 " " " " " " "

TABLE 6.—LAKE DEPRESSIONS DURING THE DRY SEASON: FOR 30 ROUND TRIPS.

Causes.	DAILY DEPRESSIONS, MEASURED IN INCHES.			
	1st month, 160 sq. m.	2d month, 140 sq. m.	3d month, 120 sq. m.	4th month, 100 sq. m.
I. Surface evaporation.....	0.2447	0.2828	0.2950	0.2400
II. Border evaporation.....	0.0066	0.0075	0.0088	0.0105
III. Percolation	0.4750	0.4450	0.4150	0.3770
IV. Water for lockages only	1.2834	1.4667	1.7112	2.0544
V. Normal waste by lockages	0.0856	0.0978	0.1141	0.1370
VI. Hydro-electric plants.....	0.2784	0.3182	0.3712	0.4454
VII. Leakage at gates and valves.....	0.1368	0.1564	0.1824	0.2180
VIII. Water used for maintenance.....	0.0490	0.0600	0.0600	0.0900
IX. Losses by false maneuvers.....	0.0296	0.0316	0.0385	0.0460
X. Losses at spillway.....	0.0780	0.0283	0.0000	0.0000
XI. Miscellaneous losses.....	0.0695	0.0804	0.0922	0.1112
	2.7366	2.9737	3.2884	3.7795
Less daily inflow.....	0.3466	0.2127	0.2884	0.4895
Net depression per day, in inches.....	2.3900	2.7600	3.0000	3.2900
Loss of level per month, in inches.....	71.70	82.80	90.00	98.70
Credit rainfall.....	0.84	0.30	0.30	1.30
Net depression per month	70.86	82.50	89.70	97.40

Total loss of level in	30 days	70.86 in.	= 5.90 ft., say	6 ft.
" " " " after	60 "	153.36 "	= 12.78 " "	12 "
" " " " "	90 "	243.06 "	= 20.10 " "	20 "
" " " " "	120 "	340.46 "	= 28.37 " "	28 "

Total loss of draft in canal during one Dry Season.

After 30 days' operation	70.86, say	6 ft.
" 60 " "	153.36 "	12 "
" 90 " "	243.06 "	20 "
" 120 " "	340.46 "	28 "

TABLE 7.—RESULTS OF ONE DRY SEASON'S OPERATIONS (30 ROUND TRIPS), STARTING WITH LAKE FULL, UP TO + 87-FT. MARK.

	Depression.	USABLE SALT-WATER DRAFT LEFT IN CANAL.			
		Absolute.	3 ft. clearance.	4 ft. clearance.	6 ft. clearance.
Beginning of dry season.....	0.00 ft.	45.60 ft.	42.60 ft.	41.60 ft.	40 ft.
After 30 days.....	6.00 "	39.77 "	36.77 "	35.77 "	32 "
" 60 "	12.00 "	33.95 "	30.95 "	30.00 "	28 "
" 90 "	20.00 "	26.19 "	23.20 "	22.20 "	21 "
" 120 "	28.00 "	18.43 "	15.40 "	14.43 "	12 "

Table 7 shows that:

After the first month of operation, the contemplated 40-ft. depth of water will have disappeared from the Canal.

After 60 days, vessels drawing 31 ft. cannot navigate the canal.

" 90 " " " 24 " " " " "

" 120 " " " 16 " " " " "

RÉSUMÉ.

A résumé of all the foregoing computations is shown in Table 4, for 20 trips a day, with the corresponding results for one dry season's operations.

As it has been claimed that the capacity of the Lock Canal could be enlarged at any time later, or whenever needed, by the construction of additional locks alongside of those under discussion, it is not idle to ascertain now whether the visible supply of water above the lake would permit such enlargements.

For that purpose Tables 6 and 7 have been prepared for 30 trips only, and assuming that, with the additional lockage, facilities could be obtained.

They show that, if additional locks were built and a double service attempted, the consumption of water would lower the lake and leave

only a 19-ft. draft. This would make it impossible to use the canal while that rate was kept up, unless some system of pumping and repumping the water back from the sea into the lake were established at an onerous and perpetual expense.

The tables prove also that the Lock Canal, by construction and by insufficiency of water, is forever limited to less than the 20 round trips taken as a basis in this study, on account of the resulting reduction of the navigable draft, if a larger service were maintained.

It is admitted, of course, that even this small rate will not be reached during the first years of operation; but the Canal is to serve forever. What the requirements of commerce will be, in years to come, is beyond an engineer's computation; history, however, shows that all ways of communication created by man have had to be enlarged, in order to satisfy the ever-increasing requirements of successive generations.

The Lock Canal, as now being built, can never be enlarged, as it could not be operated if enlarged, on account of mere lack of the necessary water.

In a sea-level canal, fed at each end by the ocean, this problem of water supply is completely and forever eliminated.

The sea-level canal can thus be at any time enlarged to meet any possible requirements of navigation in the future, for its water supply, being unlimited, need never be considered.

Deductions.

Tables 4 and 5, for 20 trips, tend to prove:

1.—That after 60 days of dry weather, the large vessels now afloat and all the men-of-war of the navy drawing more than 33 ft. cannot use the canal, on account of lack of depth.

2.—That after 90 days of dry season, all naval vessels, transports and craft drawing more than 28 ft. could not navigate Lake Gatun.

3.—That after 120 days, no vessel drawing more than 21 ft. can use the canal.

It may be argued that only the large transatlantic transports draw more than 37 ft., and that they will probably never cross the Isthmus. They are, nevertheless, now afloat; and other semi-carriers are now being completed that will draw 41 ft. on Lake Gatun.

In case of war, these and the largest transports will be the very

ones impressed for the service. If they cannot pass the Isthmus except at the end of the rainy season, an enemy could very well choose the proper time to strike us when the Lock Canal is incapacitated by mere lack of water in Lake Gatun.

Conclusions.

The foregoing deductions prove:

1.—That the water supply as planned for the Lock Canal will be insufficient and inadequate for the most important service for which the Canal is being built: the use of the United States Navy.

2.—That this insufficiency of water limits for all time the capacity of the Lock Canal to less than 20 round trips a day.

3.—That all the available water supply above Lake Gatun is not sufficient for a larger service, and thus makes impossible any enlargement of the Lock Canal in the future.

4.—That under the present plans the 40-ft. draft expected to be obtained in the canal cannot be maintained.

In closing this paper, the writer is aware of the fact that arguments and figures may be produced in its discussion which may apparently prove that his conclusions are not justified, but he wishes to record here his opinion that unless the existing supply of water is augmented by the construction of additional reservoirs and pumping stations, not called for by the present plans, the Lock Canal cannot be operated on the basis now contemplated.

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THE PURIFICATION OF THE WATER SUPPLY OF STEELTON, PA.

Discussion.*

BY MESSRS. H. V. HINCKLEY, WILLIAM R. COPELAND, AND JAMES E. LITTLE.

Mr. Hinckley. H. V. HINCKLEY, M. AM. SOC. C. E. (by letter).—The author states that one of the novel features of this plant is "the small area required —about one-fourth of that ordinarily required for a slow sand filter plant of equal capacity," and mentions on page 660,† "the very high rate of operation of the slow filters." On page 716† he gives the operating speed as 3 623 000 gal. per acre per day. The average capacity given by authorities to date for the successful operation of slow sand filters is 3 000 000 gal. per acre per day. It seems, therefore, that, as operated, the Steelton slow filters fail to realize four times the speed of successful operation of slow sand filters elsewhere. Possibly the author's meaning is that the pre-filtered water at Steelton can be put through the slow filters four times as fast as the raw water. It would be instructive if the author would clear up this point. It would also be interesting to know whether there was any reason, other than cost, for the use of coal in place of sand in the roughing filters.

Mr. Copeland. WILLIAM R. COPELAND, ASSOC. AM. SOC. C. E. (by letter).—There are several features of the Steelton filters which will attract the attention of those in charge of the operation of such plants, one being the fact that the works have been run without the supervision of a

* This discussion (of the paper by James H. Fuertes, M. Am. Soc. C. E., printed in *Proceedings* for August, 1909, and presented at the meeting of October 6th, 1909), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† *Proceedings*, Am. Soc. C. E., for August, 1909.

resident chemist. It seems remarkable that such a changeable river water can be handled without the constant supervision of a technically trained chemist, and it can only be explained by the fact that Mr. Fuertes, aided by a corps of technically trained men, bacteriologists, chemists, and engineers, made a most thorough study of the water at Harrisburg only a few years ago, and that the filter plant at Harrisburg has enabled him to study still further the conditions which prevail at all seasons of the year. Mr. Copeland.

The history of this situation teaches a most important lesson, namely, the value of thorough preliminary study. If it had not been for the experiments at Harrisburg, it is probable that the Steelton plant would not have been constructed according to the present design, and it is doubtful whether it would have been possible to operate it, at least for some years, on the present economical basis.

Table 4 shows that the plant treats a water having an average turbidity of about 300 parts per million, by using an average of only 1 grain of coagulant per gallon—a quantity hardly more than half as much as that used to treat the turbid Ohio and Mississippi waters when they carry equal amounts of turbidity.

The economical application of coagulant was made possible, and was controlled to a great extent, by the fact that Mr. Fuertes, having at hand the data secured at Harrisburg, prepared the diagram, Fig. 5,* showing the proper relation between the turbidity of the river water and the number of grains of coagulant required. This diagram presents the rather unexpected feature that less coagulant is required for a given turbidity when the river is rising than when it is falling. The author explains this by the fact that the particles which form the turbidity in a rising flood are coarser than those in a falling flood, and settle out more rapidly; moreover, high numbers of bacteria are often found in water of low turbidity, thus requiring extra coagulant.

It is stated that the river sometimes carries a natural coagulant, and to this fact, in large degree, is attributed the economical use of alum. The writer's experience with the Allegheny, Monongahela, and other waters of Western Pennsylvania which carry mine drainage, is that the first flood run-off sweeps large volumes of the acid mine waters into the rivers, or out of the pools in the tributaries, and into the main streams, thereby adding sulphate of iron from the coal-pits and washeries to the mountain waters charged with lime. As Mr. Fuertes points out, the alkalies precipitate the iron salts in fluffy clots which absorb and precipitate the mud, clay, bacteria, etc., carried by the storm-waters from the hills and fields. So potent are these iron salts as clarifiers that the usually turbid Allegheny has been made as clear as crystal for 40 miles, and the bacteria have been reduced from 5 000 or 6 000 per cu. cm. to 2 per cu. cm. Moreover, the acid mine waters

* *Proceedings*, Am. Soc. C. E., for August, 1909.

Mr. Copeland. have such a poisonous effect on organic life that they kill thousands of fish at such times.

These conditions are often accompanied by another peculiar feature, known to river men as "clear-water" rises. These rises, of course, are nothing more than the flood-waters clarified by the precipitation of the mine drainage. These clear waters start at a place where some acid water enters, and are marked by clumps of brown hydrate whirling around in spots about the size of a bushel basket.

The acid carried by the water at such times neutralizes the lime carbonates in the rivers to the extent of rendering the water acid. The writer once saw a band of Italian laborers driven out of the Kiskiminitas, where they had been putting up a bridge pier, by the acid mine drainage swept out of the Loyalhanna by a thunderstorm.

As the coal mines are being extended every day, it behooves all engineers who are planning water purification works along streams in mining districts to provide means for adding extra alkalies, if need be, to neutralize the acid mine wastes.

Mr. Fuertes has made such provision at Steelton, and attention is called to the large drain pipes under the lime tanks. Large drains are very necessary wherever lime solutions are to be made, and plenty of blow-offs should also be provided. The lime solutions do not corrode iron, but form crystals of carbonate, which, accompanied by the insoluble sludge, in the course of time, will clog the pipes.

While discusssing the subject of coagulants it should be noted that plans have been made to reduce the turbidity of the effluent from the roughing filters at all times to less than 25 parts per million. Engineers in charge of the operation of filter plants have been learning for years that the secret of securing a good effluent from a slow sand filter lies in a thorough preliminary treatment. The cleaner and purer the raw water, the longer the slow filters will run. Ten years ago it was said that a slow sand filter could handle water having a turbidity of 50 parts per million—that is a fact, but it can do so only for a short time and at rates of filtration of less than 3 000 000 gal. per acre per day.

Inventors have spent thousands of dollars and have tested almost every kind of material, from the filamentous sponge clippings and felts made of asbestos or wood fibers, to coke, gravel, slag, etc., but, for the most part, all have proved failures. The reason is plain. One cannot clean such beds without taking them apart, a process which is laborious, slow, and expensive. Unless this is done the beds become clogged so that the water is forced to rush through in constricted areas, at the same time scouring out part or all of the mud which was deposited in the bed during an earlier part of the run.

Mr. Fuertes has taken this matter into careful consideration, and has built his roughing filters out of particles which have an effective size of only about 1 mm. These grains are so small and light that the

currents of wash-water applied will suspend every grain and separate it from its neighbors. Mr. Copeland.

Unless the roughing filter will reduce the turbidity of the applied water to 25 parts per million, it is essential to add a coagulant. Where lime and iron are added, the best results can be obtained by applying them as near together as possible, so that the lime is brought into contact with the iron while the former is in the condition of caustic lime (CaO) and not as the mono- or bi-carbonate of lime.

Where the water is first subjected to chemical treatment, for the purpose of "softening," the lime added will often precipitate magnesium in flocculent masses, and these flocs make most excellent coagulant. In fact, the preliminary treatment at a water-softening plant will often remove from 75 to 80% of the suspended matter and 90% of the bacteria by sedimentation. The slogan of an efficient preliminary treatment, therefore, must be: Reduce the turbidity of the raw water to 25 parts per million and the bacteria to less than 200 per cu. cm.

JAMES E. LITTLE, ESQ.*—This paper contains an unusually clear and complete statement of the conditions existing at Steelton and the methods being followed to secure a pure water supply. As might be expected, the citizens of the borough are justly proud of the good showing the filter plant is making, and feel that they owe no small debt of gratitude to its able designer, who has spared no effort to make this plant a success. Mr. Little.

In order to corroborate the author's statements, and as a supplement to the paper, some additional data concerning the operation of the plant during its first year are submitted.

The plant has done far better than was expected in the matter of cost of operation, without any sacrifice in efficiency. No material alterations have been found necessary, which fact is notable when the novelty of the design is considered. At present sufficient laboratory equipment is being purchased to enable the superintendent to make bacteriological and microscopic determinations, after which a daily examination of the water will be made.

A form of graphical report of the operations has been devised, which is made up monthly on thin paper. Blue prints from this report are sent to the various officers of the borough who are interested in the operation of the plant. These, when properly pasted together, form a continuous graphical report which has been found of considerable value in comparing operations at different periods. The daily averages of height of river, turbidity of raw water, quantity of chemical dosing, gallons of water filtered, and gallons of water used in washing prefilters are plotted to convenient scales, and the monthly averages are given in every case. The reports of the bacteriological analyses are placed at the top of each sheet.

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*Member, Board of Water Commissioners, Steelton, Pa.

Mr. Little. The average condition represented by this record, which covers the period from January to September, 1909, inclusive, is as follows:

Height of river above low-water mark, in feet..	3.38
Turbidity of raw water, in parts per million...	87.50
Chemical dose, in grains of alum per gallon...	0.237
Water filtered daily, in gallons.....	2 092 000
Water used daily in washing preliminary filters, in gallons.....	38 960

The latter figure indicates that the length of the average run of each filter during this period was approximately 24 hours, as 13 560 gal. are required for each washing.

As stated by Mr. Fuertes, no alum was used until January 9th. From this date until May 15th the dosing was continued in varying proportions, except on nine days. From May 15th to the end of September no alum has been used, except on six days in June. The average dose of alum for a full year's operation was 0.157 grain per gal. This is considerably less than the quantity noted earlier in the year and quoted by the designer. It is expected, however, that further reductions will be made in this particular when more frequent analyses of water are possible.

The efficiency of operation—represented by the percentage of removal of bacteria—has been averaged for each month with the following result:

	Percentage removed by preliminary filters.	Total percentage removed.
January	82.38	99.015
February	96.73	99.74
March	89.36	98.98
April	93.14	98.90
May	90.42	99.67
June	91.81	96.49
July	95.82	97.91
August	97.72	97.86
September	90.42	96.78
Averages.....	91.98	98.37

These figures are not as reliable as they might have been, as analyses were made only once a week, but they give a good idea of the results obtained. After all, a satisfactory report from the borough health officer gives the best idea of the filtration efficiency. During the first year of operation not a single case of typhoid fever was reported, which is an unusual record for Steelton. Unfortunately, during the past few days, some cases have developed, but most of these are readily traceable to outside causes. A Labor Day picnic on a nearby island, when raw river water was used for drinking purposes, is undoubtedly responsible for four cases.

Mr. Little.

During certain periods of the year, when flood conditions obtain, the character of the water has to be watched very carefully. A notable instance of the suddenness of changes is recorded for January 26th and 27th, when the turbidity increased from 25 to 1 500 parts per million in less than 7 hours. At such times it has been found best to regulate the chemical dose so as to produce an effluent from the roughing filters as nearly constant in character as possible, rather than follow the ordinary rules. Two charts have been prepared, showing the details of operation from April 15th to 19th and May 1st to 5th, which illustrate the necessity for this. In each case the increase in turbidity was accompanied by a change in the character of the water, the high turbidity being caused by yellow clay having a coefficient of fineness of less than one. This was not handled with entire success the first time, but a record of the second period shows that there is no serious difficulty in removing the turbidity entirely from such troublesome water in preliminary filters and producing an effluent which will vary but little from zero. In fact, it has been demonstrated that an actual zero effluent can be maintained, if proper care is taken.

Regarding the cost of filtration at Steelton: Unfortunately, the accounts are not kept in such a manner as to permit making a comparison of the details of costs with those estimated by the designer. The following statement of totals, however, will give an indication of the results obtained. The cost of operation given includes everything that could be legitimately charged to this account, namely, superintendence and professional service, regular labor and attendance, extra labor, analysis, chemicals, light, heat, power, insurance, repairs, and minor improvements. In order to make a comparison with the designer's estimate, the cost of operation has been estimated on the basis of 3 000 000 gal. daily, by adding the cost of additional chemicals and power required, and by adding proportionally to the charge for scraping the slow sand filters. It is believed that all other charges would be unchanged by an increased output.

	Total operating cost.	Cost per million gallons filtered.	Estimated cost per million gallons, on basis of filtering 3 000 000 gal. per day.
Sept. 12th, 1908, to Feb. 28th, 1909	$1 477.65	$4.505	$2.80
Mar. 1st, to Aug. 31st, 1909.....	1 597.06	4.232	3.027
	$3 074.71	$4.359	$2.914
Value of wash-water used @ $10 per 1 000 000 gal.....	180.88	0.025	0.025
	$3 255.59	$4.384	$2.939

The slow sand filters have been scraped only once during the year. The cost of this scraping was $50.41, which is at the rate of $0.068 per 1 000 000 gal. filtered.

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AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

THE TRACTION OF FREIGHT TRAINS AT DIFFERENT SPEEDS.

Discussion.*

By A. C. Dennis, M. Am. Soc. C. E.

Mr. Dennis.

A. C. Dennis, M. Am. Soc. C. E. (by letter).—The following formula was determined by the writer after considerable experimenting with freight trains consisting of box cars of variable loading:

$$R = 2.6\ W + 85\ N,$$

in which R = Resistance, in pounds, for box cars on a level tangent.

W = Gross weight, in short tons, of the train behind the tender.

N = Number of cars in train.

Both factors vary with the class of track, rolling stock, weather, and many other minor influences, but have remained remarkably close for all test trains run by the writer.

This resistance may be considered as doubled for starting, but constant for all practical purposes after a train is in motion. Speed, wind, and temperature have little effect. The increased journal resistance which occurs when a train stands for some time is very greatly increased by extreme cold.

It is believed that ultimately freight train resistance formulas for practical rating will take the above simple form, the coefficients applicable being determined by experiment.

* This discussion (of the paper by Clinton S. Bissell, M. Am. Soc. C. E., printed in *Proceedings* for September, 1909, and not presented at any meeting), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

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PAPERS AND DISCUSSIONS

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THE USE AND CARE OF THE CURRENT METER, AS PRACTICED BY THE UNITED STATES GEOLOGICAL SURVEY.

Discussion.*

By Messrs. Arthur H. Diamant, E. C. Murphy, and H. K. Barrows.

Mr. Diamant.

Arthur H. Diamant, Assoc. M. Am. Soc. C. E. (by letter).—Too much cannot be written on the value of current meter gauging work, and Mr. Hoyt is to be commended for this paper. The writer has been in charge of gauging the New Croton Aqueduct of New York City for the past seven years, and thinks that a description of the gauging station and the method of making the gaugings may be of interest.

The station is located at Dunwoodie, N. Y., and is a circular shaft tapering from 13.5 ft. in diameter at the aqueduct to 6 ft. in diameter at the surface of the ground, with a height above the crown of about 10 ft. A portable frame house is placed over the opening for the protection of the observers.

A bronze plate or limb, grooved and graduated as shown on Plate XCVI is fastened to the side, and a revolving sleeve is attached near the top of the shaft for the purpose of holding and guiding the meter rod. This rod is a brass tube, $1\frac{1}{16}$ in. in outside diameter. It is fitted with collars and lugs, spaced as indicated on the drawing, and lettered to correspond with the grooves on the graduated limb. The area of cross-section of the aqueduct has been divided into areas of 1 sq. ft. each, and the revolving sleeve and graduated limb have been designed and placed so that the position of the meter is exactly known in each square

*This discussion (of the paper by John C. Hoyt, M. Am. Soc. C. E., printed in *Proceedings* for August, 1909, and presented at the meeting of September 15th, 1909, is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

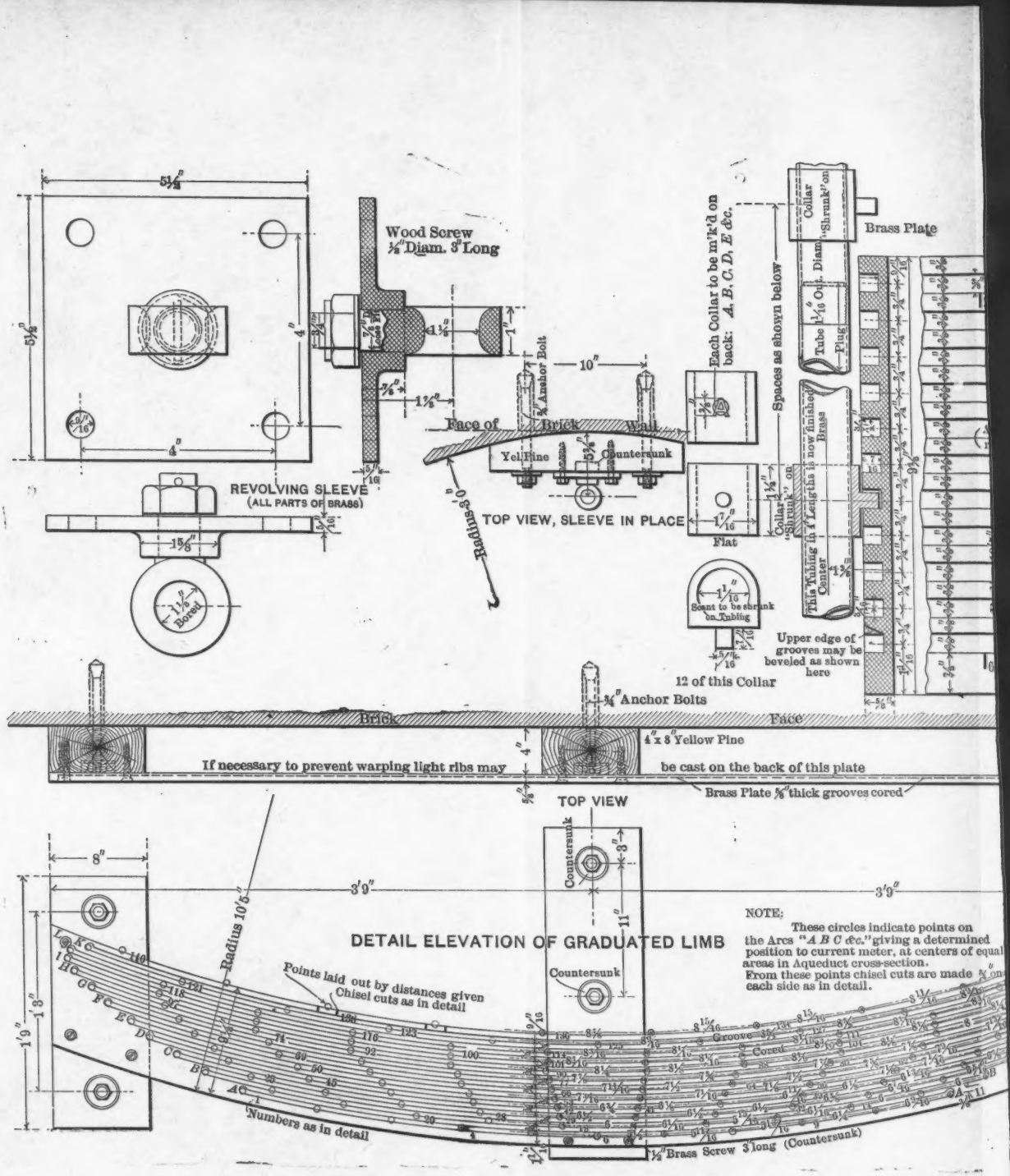
Mr. Diamant. foot of wet area. Plate XCVI shows a detail of the bronze plate, the rod and collars, the revolving sleeve, and also a general view of the gauging station with the limb in place and the meter in the first position. To prevent the rod from being bent down stream, a guide line (a cod-fish line) was attached about 1 ft. above the meter, the other end being fastened to a pole lying across a platform erected near the crown of the aqueduct for the men holding the meter rod in position.

The current meter used is a Fteley and Stearns instrument, the wheel being 4 in. in diameter. During the past year a heavier instrument was used, the wheel being 5 in. in diameter. Before the writer took charge, the observations were read on dials geared to the wheel, the dial plate being thrown in and out of gear by a spring attachment. This method caused much loss of time, as the spring did not work satisfactorily when the meter was under 5 ft. of water, with a velocity of 2 or 3 ft. per sec., and as the meter had to be drawn out of the water after each observation. Very often the meter still recorded on being pulled up, thus spoiling the observation.

The writer after having used the instrument in this condition, recommended that an electrical recording device be attached, which was accordingly done, thereby cutting down the time of gauging the aqueduct to one-half. A small cam attached to the recording wheel strikes a brass spring at every revolution of the vane wheel, thus making and breaking the electrical circuit for each revolution of the meter. An automatic counter recorded the revolutions when more than 100 per min., but for less than that number a sounder was thrown into the circuit, and the revolutions were actually counted. No trouble has been experienced, the whole apparatus working perfectly.

Current meters should be rated frequently, and as the Aqueduct Commissioners had no rating station, the meter had to be sent to Lawrence, Mass. As this occasioned much delay, a rating station was built at Dunwoodie, N. Y., on the writer's recommendation. A paved blow-off channel, 20 ft. wide and 600 ft. or more long, had been built in connection with the aqueduct, and, being in a valley, made an ideal place for the station. This consists of a timber dam with gates, for the purpose of flooding the channel to a depth of from 4 to 5 ft., a wooden platform 180 ft. long, carrying the track, and a car for holding the meter and rod. Fig. 1, Plate XCVII, shows the car, the current meter and rods, the electrical recorder and sounder, and the automatic switch which throws the recording mechanism in and out of the circuit. Fig. 2, Plate XCVII, shows the method of making the rating.

For a rating, the length of the course used was 150 ft., and about 50 observations were taken varying from about 0.5 to 8 ft. per sec. The car was pushed forward and backward over the course, to counteract any possible current. It was found that the friction increased so rapidly for velocities less than 0.5 ft. per sec. that the rate curve


Wood Screw
⅛" Diam. 3" Long
Face of Brick Wall
Yel. Pine
Countersunk
¾" Anchor Bolt
10"
Each Collar to be m'k'd on back: A, B, C, D, E &c.
Spaces as shown below
Collar "Shrunk" on Tube
Tube 1 1/16 Od. Diam. "Shrunk" on Plug
Brass Plate
REVOLVING SLEEVE
(ALL PARTS OF BRASS)
TOP VIEW, SLEEVE IN PLACE
Radius 3'0"
Flat
Collar "Shrunk" on Tubing
This Tubing is 1' Length is now finished Brass
Scant to be shrunk on Tubing
Upper edge of grooves may be beveled as shown here
12 of this Collar
¾" Anchor Bolts
Brick Face
4" x 8" Yellow Pine
If necessary to prevent warping light ribs may be cast on the back of this plate
Brass Plate ⅝" thick grooves cored
TOP VIEW
Countersunk
DETAIL ELEVATION OF GRADUATED LIMB
Radius 10'6"
Points laid out by distances given
Chisel cuts as in detail
Numbers as in detail
Groove
Cored
NOTE:
These circles indicate points on the Arcs "A B C &c." giving a determined position to current meter, at centers of equal areas in Aqueduct cross-section.
From these points chisel cuts are made ⅛" on each side as in detail.
½" Brass Screw 3" long (Countersunk)

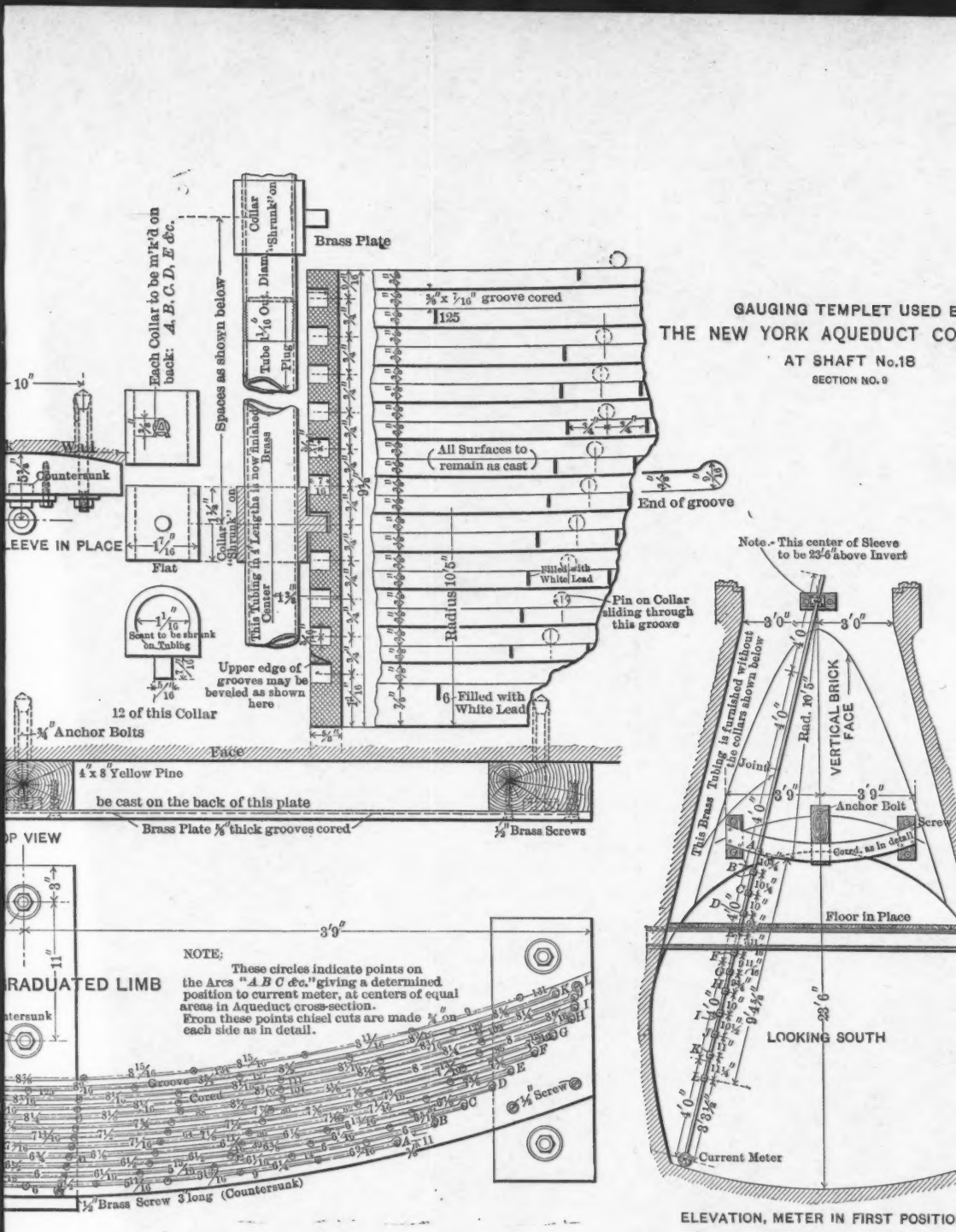
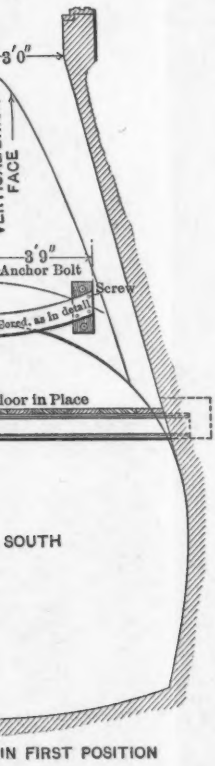
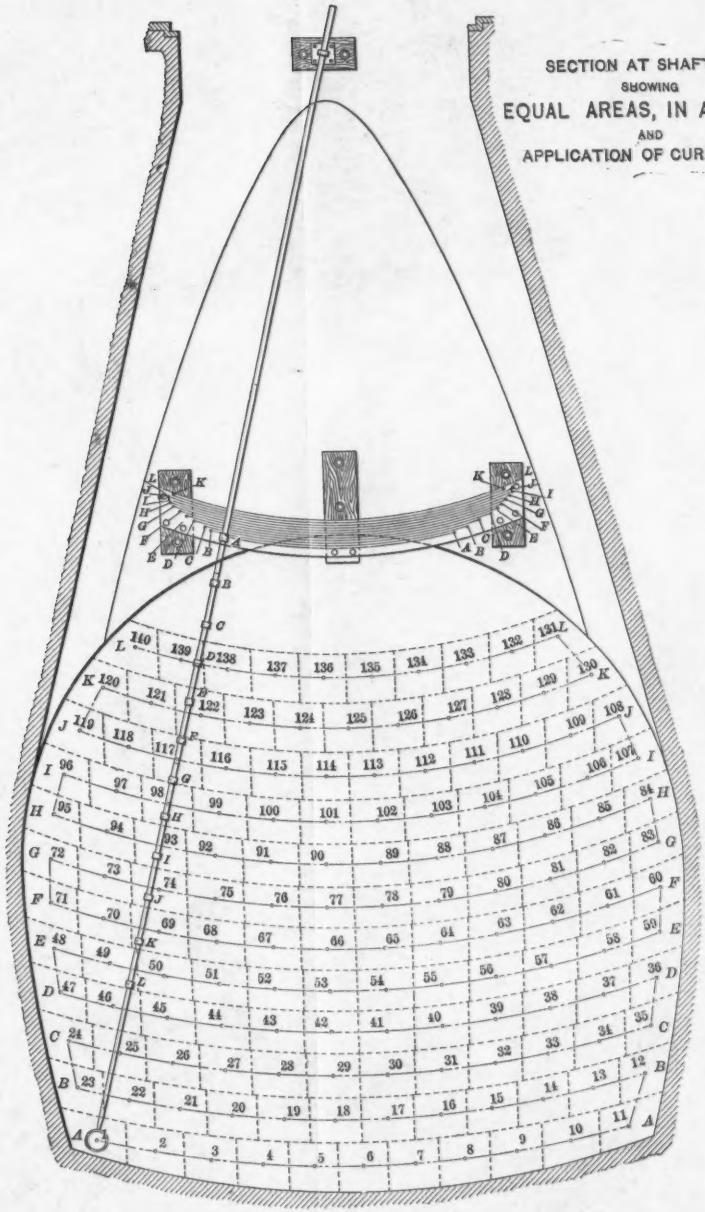

GAUGING TEMPLET USED B
THE NEW YORK AQUEDUCT CO
AT SHAFT No.18
SECTION NO. 9
Brass Plate
Collar "Shrunk" on
Each Collar to be m'k'd on back: A. B. C. D, E &c.
Spaces as shown below
Tube 1 1/16" Out. Diam.
Plug
3/8" x 7/16" groove cored
125
All Surfaces to remain as cast
End of groove
Note.- This center of Sleeve to be 23'6" above Invert
10"
Wall
Countersunk
LEEVE IN PLACE
Flat
1 7/16"
Collar "Shrunk" on
This Tubing is now finished Brass
Center
Radius 10'5"
Filled with White Lead
Pin on Collar sliding through this groove
Scant to be shrunk on Tubing
Upper edge of grooves may be beveled as shown here
12 of this Collar
3/4" Anchor Bolts
Face
4" x 8" Yellow Pine
be cast on the back of this plate
Brass Plate 3/8" thick grooves cored
1/2" Brass Screws
3'0"
Joint
Rad. 10'5"
VERTICAL BRICK FACE
This Brass Tubing is furnished without the collars shown below
3'9"
Anchor Bolt
Screw
Cored, as in detail
Floor in Place
23'6"
LOOKING SOUTH
Current Meter
ELEVATION, METER IN FIRST POSITIO
OP VIEW
RADUATED LIMB
NOTE:
These circles indicate points on the Arcs "A B C &c." giving a determined position to current meter, at centers of equal areas in Aqueduct cross-section.
From these points chisel cuts are made 3/4" on each side as in detail.
Groove
Cored
1/2" Screw
1/2" Brass Screw 3" long (Countersunk)

PLATE XCVI.
PAPERS, AM. SOC. C. E.
OCTOBER, 1909.
DIAMANT ON
THE USE AND CARE OF THE CURRENT METER.

MPLET USED BY
QUEDUCT COMMISSION
AFT No.18
ON NO. 9

SECTION AT SHAFT 18
SHOWING
EQUAL AREAS, IN AQUEDUCT
AND
APPLICATION OF CURRENT METER





IN FIRST POSITION

approximated a hyperbola. For a speed greater than 8 ft. per sec., the electrical recorder would not count properly. The method of taking observations was identical with that described by Mr. Hoyt. Mr. Diamant.

The writer found that the ratings varied but little, and that for this Fteley meter, the rate curve was a straight line between the velocities, 1.2 and 8 ft. per sec., but for velocities of less than 1.2 ft. per sec. it was a hyperbola. Accordingly, in calculating the rate equation, he used the formulas $b \Sigma x + n a = \Sigma y$ for the straight line, and $a \Sigma x + b \Sigma x^2 = \Sigma x y$ for the curved part, where $x =$ the velocity, in feet per second, and $y =$ the number of revolutions per second. The observations were also plotted and a curve was drawn, which agreed very closely with the calculated curve.

E. C. MURPHY, M. AM. SOC. C. E. (by letter).—This valuable paper will be read with much interest by hydraulic engineers, especially by those who measure the discharge of streams or who consult the stream-gauging data of the Water Resources Branch of the United States Geological Survey. Mr. Murphy.

The work being done by the engineers of this Branch, to simplify the current meter and to adapt it to the measurement of stream discharge, is certainly very commendable, and a comparison of their instruments and methods with those used in other countries cannot fail to show this fact.

The author's clear-cut description of the small Price meter and its accompaniments and his remarks on the care of the current meter are very good.

The writer is specially interested in the data of Table 3, showing the percentage of deviation of ratings of nine current meters from that of a mean rating of those meters. This comparison shows a close agreement in the behavior of meters of this type when in good condition and when operated in exactly the same way. In measuring the discharge of a stream, it is not always possible to have the current meter in exactly the condition it was when rated, nor to hold it in the water in the way it was held when rated, therefore a table showing the percentage of deviation of rating of a meter under various conditions which can be described accurately with a rating under standard conditions would be very valuable. With the data of such comparative ratings, an estimate can be made of the error due to conditions not being standard. The writer suggests that the meter be rated under the following conditions:

1.—With dull point,
2.—With one or more cups slightly damaged,
3.—With meter not well oiled,
4.—With water near freezing point,
5.—With weights near to cups,

Mr. Murphy.

6.—With cups close to bottom of channel,
7.—With cups close to water surface,
8.—With meter tipped upward or downward,
9.—With meter held with a rod not free to tip,
10.—With meter held with a rod free to tip.

The effect of a Pitot tube on the velocity of the water near the bottom of a channel has recently been observed by the writer in the U. S. Geological Survey Débris Investigation Laboratory. The channel was a trough with glass sides, carrying mixtures of water and sand. Although the part of the Pitot in the water had a width of less than $\frac{1}{8}$ in. as it approached the sand bed, the sand scoured from under it leaving a space between the lower end of the Pitot of from $\frac{1}{4}$ to $\frac{1}{2}$ in., depending on the velocity. If a velocity-measuring instrument acts thus on the bed of a stream, the bed re-acts on the instrument and may affect its rating.

Mr. Barrows.

H. K. Barrows, M. Am. Soc. C. E.—Several interesting questions have been suggested by this discussion. As far as the speaker knows, the rating of current meters in running water has never been attempted. While this experiment would be interesting, from a scientific viewpoint, it hardly seems necessary in a practical way, and the rating ordinarily made in still water is applicable to the ordinary case of stream measurements with no appreciable error. Further, comparisons of the results of current-meter measurements of discharge with the computed discharge over standard weirs, have been made in a sufficient number of cases to establish the substantial accuracy of the current meter when rated as described in this paper.

More investigation along the lines of practical hydraulics is needed, both in the use and improvement of such instruments as the current meter and in the derivation of suitable coefficients in other methods of estimating discharge. For example, in the use of gates to allow the discharge of stored water from lakes and ponds, little or no information is available on which to base the discharge under different conditions. To some extent the engineers of the United States Geological Survey are procuring information along such lines, but, with the limited funds available for stream gauging, it is obvious that little or no experimental work is possible. The speaker would urge more practical investigations on a full-sized scale by the universities and colleges equipped for hydraulic work, and the prompt publication of results.

The matter of coefficients for reducing surface velocity to the mean velocity in the vertical, as discussed by Mr. Grunsky, is important, as during the time of the year that, owing to the swiftness of the current, only surface velocities can be obtained, a considerable percentage of the total yearly run-off occurs, and errors in flood measurement may

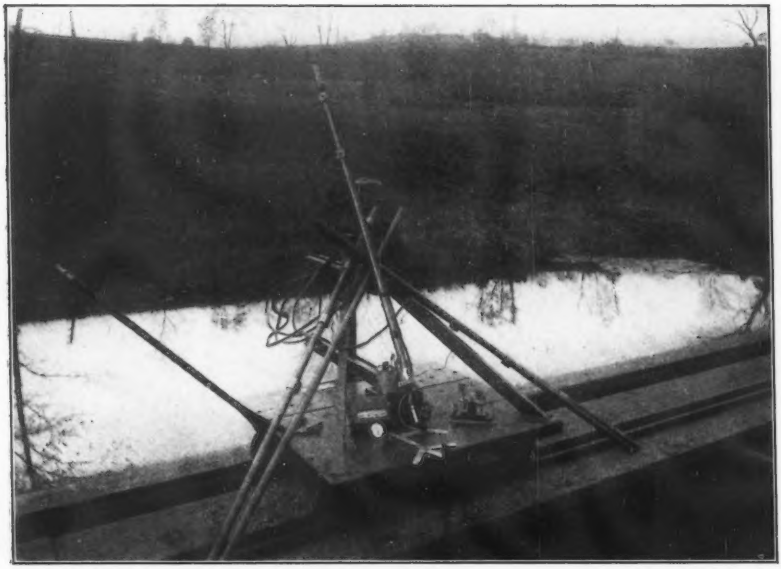


FIG. 1.—CAR, CURRENT METER, RODS, ETC., NEW CROTON AQUEDUCT GAUGING STATION.

FIG. 2.—METHOD OF MAKING RATING, NEW CROTON AQUEDUCT GAUGING STATION.

Mr. Barrows.

have serious results in the estimation of the quantity of water available for storage. This is one of the many problems of stream gauging studied by the U. S. Geological Survey, and for which satisfactory methods and coefficients have already been derived. With the limited appropriation for carrying on the work, however, and the uncertainty of the continuation of even this amount from one year to the next, it is not possible to maintain a full corps of competent men to carry on field work, so that some errors in results must be expected under present conditions. If a suitable appropriation were insured, extending over a period of at least ten years, many of the present sources of error could be avoided, and the work of stream gauging could be carried on continuously, in the broad and comprehensive way that it should be.

The speaker would urge especial care in the use of the current meter under such conditions as are met in head-bays, tail-races, etc., where the conditions of flow are governed by the manner in which water is being used, and may vary greatly during short intervals of time. The usual methods for the determination of mean velocity by current meter are not applicable, and, in general, the multiple-point or vertical-curve method is the only one which will give good results. The failure of the Price meter to give correct velocities where there are many cross-currents and much "boiling" must also be kept in mind, and a suitable section, reasonably free from such conditions, must be chosen for the observations.

The "two-tenths-and-eight-tenths-depth method," for the determination of the mean velocity in the vertical, has come to be the standard, both for open water and for ice cover. This has been of especial interest to the speaker, as it was first suggested by him in 1905, as being a most promising method for the quick determination of mean velocity under all conditions.* This formula was developed in the study of vertical-velocity curves under ice cover,† and was found to apply even more truly to flow in open channels. It accords with the assumption that the typical vertical-velocity curve is a common parabola with horizontal axis. Some time after this formula was suggested by the speaker in 1905, he learned that, at a previous date, Thomas U. Taylor, M. Am. Soc. C. E., had concluded that, theoretically, the mean of observations at 0.21 and 0.79 depth should give the mean velocity in the vertical. The speaker would be interested to learn whether this method was ever in practical use prior to his adoption of it for work in New England.

An important point in respect to current-meter measurements under ice cover is that a single observation at 0.5 depth with the application of a coefficient ranging from 0.82 to 0.92, with a mean of 0.88, gives a quick and fairly reliable determination of the mean velocity in the

* *Journal of the Association of Engineering Societies*, July, 1905, pp. 19–20.

† Water Supply Paper No. 187, U. S. Geological Survey, pp. 84–85.

Mr. Barrows. vertical. This may be called the single-point method for ice cover, just as the six-tenths-depth method is for open water.

In concluding, the speaker wishes to emphasize the fact that criticisms and suggestions regarding the work and methods of the U. S. Geological Survey are welcomed at all times by its engineers, and that an effort is made to profit by them. The suggestions from practicing engineers, who make use of the estimates of stream flow, are especially valuable as, from their close study of these data, they realize just how improvement can be made.

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AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

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THE REDEMPTION OF THE GREAT VALLEY OF CALIFORNIA.

Discussion.*

By William W. Harts, M. Am. Soc. C. E.

Mr. Harts.

William W. Harts, M. Am. Soc. C. E. (by letter).—The problem of determining what treatment is best for the future prosperity of the great Sacramento and San Joaquin Valleys in California will require much patient study before anything entirely satisfactory can be evolved. The main future development of these valleys will undoubtedly be agricultural for many years, and whatever ministers most to this development as an ultimate benefit will unquestionably be the correct solution. The interests are so diverse and the difficulties of harmonizing them so great as to make the task one of enormous proportions. If the problem to be solved were the project for flood protection or for drainage of the valley alone, or if only the interests of navigation were to be considered, it seems safe to say that some definite plan could be readily devised which would satisfy the conditions fairly well. But the problem of harmonizing these important interests, difficult as it might be found, is further complicated by the necessity of providing for irrigation at certain seasons, by the improbability of securing perfect agreement among the many landholders, in order that the enormous expense may be distributed properly, and, further, by difficulties in the way of legislation, to the end that State and Federal interests may not be neglected, and, at the same time, that the charge for the necessary work be placed where it should properly belong.

Most of the lowland of this valley was given to the State of California by the General Government in 1850, under what is known as

* This discussion (of the paper by A. D. Foote, M. Am. Soc. C. E., printed in *Proceedings* for September, 1909, and not presented at any meeting), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

Mr. Harts. the "Swamp and Overflowed-land Act," with the distinct understanding that the State should take such steps as would reclaim these areas and render them fit for agricultural purposes. If this land were still "public land" of the United States, the solution of the difficulties would be far simpler. Practically all this land was sold later by the State to private parties at merely nominal figures, still coupled, however, with the obligation that it be reclaimed. In most cases this obligation has never been met, but decisions of the courts hold that the land is still subject to the call of the State for such assessments as may be found necessary for the proper solution of this great problem.

The entire central part of California consists of an elliptically-shaped valley of about 57 000 sq. miles, of which about 18 000 sq. miles are lowland, all having but one outlet, namely, through San Francisco Bay and the Golden Gate. The low part of this entire valley, it is believed, has been largely built up by the washing down of material from the surrounding mountain sides, until Suisun Bay, at its western side, is now all that remains of an enormous inland sea. It seems undeniable that the rivers flowing into this basin never have been, and probably never will be, able, in their natural condition, to carry off within their banks the floods arising from the rains and snows falling within their water-sheds; and it also seems undeniable that the sediment eroded from the high mountain sides has been carried down probably for ages and is still moving seaward, encroaching yearly on Suisun Bay. Since the discovery of gold this erosion has been aided enormously by the process of hydraulic mining, millions of cubic yards of solid material having been dislodged from the gold-bearing areas only to be transported down the stream beds into the river systems below. Thus a further problem has been added to the natural difficulties, that of mining-débris control, the solution of which is now being attacked.

It may be true, as has been stated by some, that it is an "engineering absurdity" to attempt to control the maximum flow of the Sacramento River by levees, but, notwithstanding such hard words, it is the writer's belief that in no other way can the ordinary floods of this valley ever be satisfactorily controlled, and in no other way can the delicate problems of navigation, from which much good may be expected during the future development of this valley, be solved than by some system by which practically all the river flow shall be carried in the river bed. The width apart of the levees and their height determine the quantity of water which can be controlled. The slope is practically fixed. By deepening the natural bed, navigation and drainage channels can be provided; but the proper placing of the levees must be relied on to control the high-water flow of the river. Such extreme floods as it may not be economical to provide against must occasionally find an outlet by side channels, but the main river flow must be kept within the

natural or artificial banks. Accessory works for drainage and basin irrigation may be easily coupled with this project, so that all that may be necessary in the way of flooding the side basins could be attained by the use of gates. A modified dike system, such as is proposed by the author, might be easily applied as an accessory improvement, but the main principles as given above will inevitably be found necessary in the end. Mr. Harts.

The requirements of navigation demand that deposit shall not be made in the channels of the stream. The plan proposed by the author would certainly produce such deposits. Even relief weirs in the levees cause a shoaling in the channel below, owing to the diminution in the currents in the main stream. It is readily seen that withdrawing a portion of the flow will cause a slackening of the currents below the point of the withdrawal, and deposit will surely result, to the injury of the channels. Side drainage channels will likewise be filled up by deposits therein, owing to the diminishing velocities. It is well known that the escape of floods over the banks causes the river beds to fill and the banks to build up in the case of all sediment-bearing streams.

Experience in many rivers and in many countries shows that, in order to secure the scour, to maintain not only navigable depths but adequate drainage, practically the entire flow of the river must be controlled between either natural or artificial banks. The improvement of the Po in Italy demonstrated this nearly two hundred years ago. The mouth of the Mississippi and the mouth of the Danube illustrate this point still further. It seems probable, therefore, that no project which does not include the deepening of the present bed of the river and the control of practically all of its high-water flow by levees can ever succeed. The author provides nothing for flood control. He accepts the present flood conditions as necessary and inevitable, and endeavors only to make the best of them. To anyone familiar with the high-water conditions and the slight fall of the main rivers of the valley, and the natural delay in passing floods, the problem will seem to be far from similar to that of the Nile, from which he draws his examples, and far from solution when he proposes to hold back by dikes, even longer than is done at present, the escape of the surplus water.

As to the details suggested by the author, some criticism should also be made. His suggestion of passing drainage channels under the Sacramento River by means of siphons seems particularly vulnerable. This river, flowing as it does through sedimentary deposit, would require work of tremendous expense to conduct side channels under its bed safely, and the probability is not remote that such siphons could be kept open only with the greatest difficulty. The available head must necessarily be slight. This is obvious when the great difficulty of drainage rests primarily on the lack of fall between the floor of the valley and mean tide level. Furthermore, the water comes charged

Mr. Harts. with sediment which would unquestionably be deposited in these siphons unless expensive apparatus were devised to maintain the flow.

Mr. Foote suggests that fifty years ago the conditions of depth for navigation and drainage were far better than now, and were "of immense value to the community as commerce bearers." Records show beyond argument that fifty years ago, and even before that, the depths were no better than they are to-day. The bed has been raised slightly by sediment, but levees were built for the reclamation of the land, which have corrected this, and the navigable conditions are now no worse than they were then. Charts of 1841 show ruling depths of 7 ft. at low water between Sacramento and Suisun Bay. This depth is still available. At that time a 4-ft. depth was found between Sacramento and Colusa, and is still found. In the face of many difficulties, the former navigable conditions have been maintained to the present time. If the rivers are not as important now as they were then for carrying commerce, the answer is to be found in the development of the railways. That river commerce was greater then than now is often asserted, but never verified, and is open to challenge.

Attention is invited to an oversight in the assignment of the credit for the design of the movable dam mentioned by the author as a desirable type for construction on these streams. The dam, referred to as having been designed by Mr. E. R. Cooley and used by Isham Randolph, M. Am. Soc. C. E., at the locks and power plant of the Chicago Drainage Canal, was originally invented by H. M. Chittenden, M. Am. Soc. C. E., Col., Corps of Engineers, and used by him in the improvement of the Osage River.* It is a modification of the Desfontaines drum wicket.†

Another detail requiring comment is the author's method of consolidating gravel and sand by pumping this aggregate full of cement grout. While engaged on the restraining works on the Yuba River, the experiment of pumping Portland cement grout into the bed of the river was tried with a view to consolidating it. Many experiments were tried, but all proved unsatisfactory. The first and main objection was that one had no definite knowledge of the results of his work, and could not learn without more expense than other methods would cost; the second objection was that the water encountered affected the setting qualities of the grout, through dilution or otherwise; and third, it was found impossible to force the grout to any great distance from the pipes. The experiments on the Yuba River, where conditions were peculiarly favorable, showed conclusively that no reliance could be placed on this method. The writer does not mean to say that there may not be occasions where it might prove advantageous, as a matter of fact, it has occasionally been attempted successfully; but the conditions

* *Transactions*, Am. Soc. C. E., Vol. XXXIX, p. 533.

† *Journal of the Association of Engineering Societies*, June, 1896, p. 349.

Mr. Harts.

in the Sacramento Valley are somewhat similar to those on the Yuba River, although not so favorable for this class of work, and he believes that the author is over sanguine as to the results achievable in this way.

The holding back of the débris by a succession of overfall barriers, in the plan suggested by the author, has been abandoned by the California Débris Commission after thorough trial, and is open to serious objection. This method was applied years ago to the Yuba River, and several failures have shown that overfall dams in the beds of rivers composed of mining débris cannot be maintained without undue expense. The first attempt of this kind was made by the State of California in the early Eighties. A barrier of brush, extending entirely across the Yuba River, was built at a cost of about $250 000. This work was designed by the State Engineer, with the approval of a commission of consulting engineers, of which the late James B. Eads, M. Am. Soc. C. E., whose reputation for river work was widely known on account of his connection with the improvement of the mouth of the Mississippi, was a member. This barrier did not resist the first freshet. The next attempt was made some years later by the California Débris Commission farther up stream, and was also a brush and rock dike combined with sheet-piling on the down-stream side. This dike was likewise destroyed, and later a modified design was again attempted, using brush fascines built in "cob-house" construction and filled with rock. This plan also failed. It was evident that, even with low lifts, overfall dams on beds of gravel could not be attempted in this river, and they were then abandoned by the Commission. Accordingly, a later type of barrier was designed, to be built of concrete in successive steps held in place by piles, the down-stream side in the river bed to be composed of a broad apron of heavy rip-rap, the main flow of the river to be carried through a spillway cut through solid rock at one end, so that at all stages, except at extreme high water, all floods would pass through a rock cut, leaving the dam and the bed of the stream untouched. While under construction, and before this barrier was completed, it was partly destroyed by the extreme flood of 1907, by wearing away the concrete cover, and has not yet been rebuilt. This same general method is now being followed at Daguerre Point, lower down on the Yuba, where a dam, too high ever to be overtopped, and a side spillway, will impound more than 50 000 000 cu. yd. of mining débris. The plan suggested by the author of placing various overfall barriers across this and other similar rivers is open to this objection. No overfall dam can ever be maintained in these streams except at undue expense.

Finally, the enormous cost of any such work as proposed by the author, even if it should be assumed to be effective, would probably forbid its application. In 1905 a comprehensive plan for the drainage of the Sacramento River was prepared by a board of experienced engineers skilled in river work. The cost of this project was about

Mr. Harts. $24 000 000. The question at once arose as to how this money could be secured. The Federal Government was appealed to in the hope that it would pay one-third of the amount. The State, it was hoped, would pay one-third, leaving the remainder to be paid by the landowners interested. Inasmuch as many of these landowners had purchased their lands for $1.25 per acre, and some had had even this amount refunded to them by the State, it would appear that their hope of having two-thirds of the expense of reclamation borne by others was, to say the least, not an unselfish one. It is needless to say that thus far neither the United States nor the State of California has accepted any such disadvantageous proposition. It seems clear that the first difficulties to be surmounted in any adequate plan of treatment would be a proper distribution of the expense under such legislation as would compel all the improved property to pay its proportionate share toward the improvement. That the greater burden of this work must fall on the land in question there seems no doubt, but no project can be undertaken until some just method of collection of funds for its completion has been provided for by law.

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PAPERS AND DISCUSSIONS

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A REVIEW OF CHICAGO PAVING PRACTICE.
Discussion.*

BY MESSRS. W. W. CROSBY, E. H. THOMES, AND RICHARD LAMB.

Mr. Crosby. W. W. CROSBY, M. AM. SOC. C. E. (by letter).—The writer has found this paper extremely interesting, and desires to call attention to the following points:

Under the heading, "Wearing Surface, Brick Paving," the author, in expressing his opinion regarding the allowable limits of loss in the rattler, states that "bricks which lose more than 20% of their weight are rejected." Will he kindly state whether or not, in his opinion, it is sufficient to limit the average loss to 20%, or whether an attempt should be made to prevent too great a variation between the minimum and maximum losses by specifying the maximum variation to be allowed between the extremes? The writer believes that such a limit in variation should be specified, in order to avoid lack of uniformity in the surface.

Under the heading, "Stone Block," the author states that preference is given to certain quarries which, as shown by experience, have furnished satisfactory blocks, and apparently no tests of the blocks themselves, other than that of actual use on the streets, have been, or are, required. Does this practice result in limiting competition unnecessarily, or in excessive high cost, through a combination of the operators of the preferred quarries? Possibly a test similar to the customary rattler test of macadam materials could be made on material offered for stone-block work; and, by specifying the maximum loss

*This discussion (of the paper by P. E. Green, Assoc. M. Am. Soc. C. E., printed in *Proceedings* for August, 1909, and presented at the meeting of October 6th, 1909), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

Mr. Crosby. necessary to reject a block which would wear too rapidly to be economical, and a minimum loss lower than which a block might be expected to wear rounded and slippery too easily, one might avoid conditions, which, in some localities, might be objectionable.

Under the heading, "Wood Block," the author suggests certain limits for the allowable volatilization of creosote oil. In the writer's opinion, the 5% and 15% limits at 210° and 235°, respectively, will be raised, for the present at least, on account of existing conditions, to 10% and 20%, respectively.

Under the heading, "Asphalt," the author states that:

"Asphalt, according to experience in Chicago, is not suitable for streets having the heaviest commercial traffic, such as those in 'The Loop' district, as the best pavements laid thus far have shown a tendency to wave and wear rapidly."

It would be enlightening and interesting if, where such apparent failures have occurred, a detailed statement could be made of the composition of the pavement, both as regards the properties of the bituminous material and the character and proportions of sizes of the mineral aggregate. It will be generally agreed that many failures, such as the author refers to, are due to defects in one or the other of these factors. The author seems to be of the opinion that about ten years is practically the limit of the life of asphalt pavements. The writer admits that many have failed in even much less time, but believes that, with proper materials, asphalt pavements have been and can be constructed, which will have a life of several times the period mentioned.

In reference to concrete pavements, it would be interesting if the author could state any facts concerning the cost of repairs to such pavements. He mentions that the surface has not worn well in some places, but it is not clear whether the defects have been caused by the use of improper materials or methods, or by inequalities in either. The writer's experience is that it is very difficult, if not impossible, to secure a concrete surface which will wear evenly under traffic, frequently because of the unavoidable variation in the broken stone used and in the proportions of the mixture itself.

The work of keeping such pavements in satisfactory repair is extremely difficult, as well as high in cost.

Mr. Thomes. E. H. THOMES, ASSOC. M. AM. SOC. C. E.—Highway construction and maintenance, while one of the oldest branches of civil engineering, has only recently begun to receive the serious consideration and thorough investigation by engineers which it deserves. There is still a wide field for new and advanced methods which would effect a large ultimate economy in the millions of dollars being expended for highway improvement.

Mr. Thomes.

During the summer of 1909 the speaker made a paving inspection trip through twenty of the principal cities between New York and Chicago, and a few of the impressions gained may be of some interest.

In the past few years Chicago has made notable progress in street improvements, as has been so ably presented in Mr. Green's paper.

A matter which should receive the early and careful consideration of the municipal authorities is the complete establishment of street lines and grades for the entire area of the city, so that, as various improvements are made, they may conform to a comprehensive and well-planned street system. This should include roadway and sidewalk widths, together with explicit rules for determining the auxiliary grades of the roadway crown, the gutter line, the front and back edge of the curb, the location and slope of the paved footwalk, whether of cement or stone, the slope of grass plots, the determination of grades at all these intersecting lines, the radius of the curb at intersections, the drainage, the location of trees, areaways, stoop and house lines, etc.

The following is the ordinance in New York City for street widths in future improvements:

New York City Ordinance.

The roadway widths of streets shall be such as to give a clear space between curb lines as follows:

(*a*) For streets less than 20 ft. wide and used for vehicular traffic, the width of the roadway shall correspond with the street width, less the space occupied by the curb.

(*b*) For streets having a width ranging from 20 to 50 ft. and not occupied by a railroad, the width of the roadway shall be 60% of the total width of the street.

(*c*) For streets having a width ranging from 50 to 60 ft., and not occupied by a double-track railroad, the roadway shall have a width of 30 ft.

(*d*) For streets having a width ranging from 60 to 66 ft. 8 in. and not occupied by a double-track railroad, the width of the roadway shall be one-half of the total width of the street.

(*e*) For all streets having a width of over 66 ft. 8 in., except those portions of Fifth Avenue and of Forty-second Street, Borough of Manhattan, concerning the treatment of which a resolution was adopted by this Board on December 18th, 1908, the roadway width shall be 80% of the street width less 20 ft.; provided, however, that if the street is occupied by a double-track railroad the minimum roadway width herein prescribed for such railroad shall be required.

(*f*) For streets in which there is a single-track railroad, the minimum roadway width is to be 30 ft.

(*g*) For streets in which there is a double-track railroad, the minimum roadway width is to be 40 ft.

The curb corners at street intersections where the interior angle is 30° or more, shall be turned with a curve having a radius of 5, 6, 8,

Mr. Thomes. 10 or 12 ft., this being determined for each case as the nearest of these dimensions which would represent 10% of the width of the wider street, provided, however, that in case the interior angle is less than 80° the radius shall not be less than 20% of the distance between the building line corner and the point of intersection of the curb tangents.

For intersections where the interior angle is less than 30° a tangent shall be inserted in the curb line at the corner at right angles to the line bisecting the said interior angle, and at a distance from the building line corner equivalent to the width of the wider sidewalk of the intersecting streets, the said distance being measured along the bisecting line, the curves to connect this tangent with the curb lines otherwise provided for shall each have a radius of 6 ft.

The roadway shall be centrally located between the street lines, and for streets having a width of 20 ft. or more, the remaining space on each side of the roadway shall be designated as the sidewalk.

No encroachment shall be permitted upon any roadway unless authorized by a franchise.

No encroachment of a permanent nature shall be permitted upon the sidewalk space of streets owned in fee, or shall be hereafter permitted upon an easement street, between the elevation of the curb and a horizontal plane 10 ft. above the elevation of the curb.

If Chicago's 2-ft. and 4-ft. street corner curb radii were made larger, it would improve the general appearance and facilitate vehicular traffic. On the other hand, the radius of 15 ft. and greater on the average street is likely to encourage too high a speed in vehicles turning the corner, and to endanger pedestrians crossing, also to curtail the sidewalk space.

There are many advantages in a street system having the four intersecting street curb corners all at the level of the established grade, with a straight curb grade between adjoining streets or only one intermediate change of grade, eased off by a vertical curve and with the proper grade to drain well. The depth of gutter may then be uniform for all pavements, the crown or elevation of the center of the street being varied to meet the various conditions. In the case of steep or very flat grades, skew intersections, car tracks, permanent improvements, etc., it becomes a difficult problem to solve satisfactorily.

The most common form of street surface is the parabolic curve, with a crown of $\frac{1}{60}$ of the width of the roadway, and a 5-in. or 6-in. depth of gutter. In recent years the tendency has been to increase the crown of sheet asphalt and other pavements from $\frac{1}{100}$ and $\frac{1}{80}$ to $\frac{1}{72}$ or even $\frac{1}{50}$ of the width of the roadway. Smaller and more frequent sewer inlets are constructed, and the depth of the gutter is reduced.

Stone curbing should be set in concrete made with small stone, but it appears that an unnecessary depth of curb and an undue quantity of concrete is used in some places, the curb being 24 in. deep, with 6 in. of concrete all around. It would seem that a depth of 16 or 18 in. and 4 or 5 in. of concrete below and around the curb would be

Mr. Thomes.

ample in most cases. In wet subsoil, a drain of cinders, gravel, broken stone, or tile should be placed below the curb and under the pavement where necessary. These drains should be connected with the sewer inlets at every low point. In sandy or other well-drained subsoils, and in closely built-up sections with deep cellars and sewers, sub-drainage is not needed.

The character of the curb will depend on local conditions. Concrete curb is favored in many places on account of its cheapness and pleasing uniform appearance. A metal-edge reinforcement is an improvement, but as most of those on the market are patented and are expensive, its use is not justified in some cases. A metal-edge reinforcement on both concrete and stone curbs is advisable on street corners where the traffic is heavy. Care should be taken not to bend the metal nosing, as it is difficult to keep it to the true curb line.

The combined concrete curb and gutter is satisfactory in some suburban districts, or with a macadam or a bituminous pavement, but it does not meet with general favor in large cities where the traffic is heavy. It should be constructed in sections of about 6 ft., of well-rammed fine concrete on a firm cinder base, and should have a wearing surface finish of mortar composed of fine granite screenings or other hard grit. Considerable combined concrete curb and gutter has been laid in Chicago, and has been fairly satisfactory. It is cheaper than stone curb; it reduces the area of pavement to be paid for; it is easier to construct a smooth and varying gutter line; and it avoids the disintegrating effect of water in an asphalt gutter; but after a few years under heavy traffic it wears out at the junction with the pavement, and it is very costly to repair the pavement or the gutter.

If more attention is paid to draining, rolling, compacting, and surfacing the subgrade, a considerable saving may be effected in the quantity or quality of the concrete or stone foundation and in the maintenance of the wearing surface. In Cleveland, which probably has as good pavements as any city in the United States, special care is taken in puddling, rolling, and preparing the subgrade. The subgrade and the sand bed for block pavements are brought to the proper surface by the use of a right and a left iron-faced wooden templet moved by iron handles. The templets are about half the width of the roadway. They have a 4 by 4-in. notch in each end, which fits over a 4 by 4-in. timber placed to grade along the center of the street and along each curb. In Rochester a trussed templet, the full width of the roadway, is drawn along on the curbs, with the bottom of the templet at the proper elevation to give the true contour. In other cities the proper depth of concrete, sand bed, etc., is obtained from ordinates below a string stretched across the curb, or from the engineer's stakes, or by tees, or by stakes set at close intervals by these methods. Instead of using wooden stakes, it would seem advisable to use flat-headed iron spikes which can be easily driven, pulled up, and used over and over again.

Mr. Thomes. Too much care cannot be taken to obtain uniform depth and a true curvature of the different courses.

While a rolled-stone macadam or gravel base foundation is satisfactory in many places, the general practice in most of the larger cities is to use a concrete foundation. The good results from a stone foundation are probably due to the increased rolling. If the subgrade is properly rolled and drained, and the concrete is well compacted, better results will probably be obtained from concrete. A cement-grouted rolled-stone foundation similar to the "Hassam" pavement, would seem to offer some advantages. A roller-tamped stone foundation, similar to the improved "Petrolithic" pavement, may be suitable in many places.

Brick pavements, while quite popular in the Central States, have not been used very extensively in New York City. They have not proved satisfactory under the heavy traffic, and they cost more in this locality than asphalt pavements which give better satisfaction. Where traffic can be kept off, a good cement grout filler is preferred. Tar filler is used more than asphalt filler in most places, because it is cheaper. A mixture of tar and asphalt might prove more suitable in many cases, all conditions considered. With grouted joints, an expansion joint along the curb is advisable in practically all cases; but, in the case of frequent transverse tar or sand expansion joints, the remedy is worse than the disease, as the pavement begins to disintegrate at all these joints in a short time, and the results appear to be more objectionable than the fine cracks or occasional wide cracks which may develop where no transverse expansion joints are used. In most cases the cross expansion joints are not advisable, especially for stretches of less than 1 000 ft., or where the bricks are heated and expanded, as they usually are, when laid in the summer. Where provision for expansion is deemed necessary, the wood block expansion joint, as developed in Detroit in 1908, gives promise of being more satisfactory than the usual bituminous transverse joint. Two courses of creosoted, 4-in., long-leaf pine, paving blocks are laid across the street about every 100 ft. They are laid, rolled, grouted, and treated in exactly the same manner as bricks. They have been in use for more than a year, and seem to take care of the expansion without developing any noticeable cracks, either at or between the wood blocks. The wood blocks will probably wear about as well, if not better, than the adjacent brick, as they are likely to wear more uniformly than the brick.

While the speaker was in Chicago, the wood block pavement in front of the Auditorium on Michigan Avenue, south of Van Buren Street, referred to by Mr. Green, was being removed to permit the widening and change of grade of the new Michigan Avenue (Park) Boulevard. Only a small strip of the wood block pavement remained, but it was in excellent condition, and the blocks showed a wear of only about

¼ in. in the ten years it had been laid. Further authoritative detailed information about this piece of pavement would be very desirable. It was laid on a mortar bed, which is the New York City practice. Most of the wood block pavements in the West are laid on a sand bed and rolled like brick. In the latter case the general appearance of the surface is usually better, due to the rolling, but a mortar bed does not shift, and it holds the blocks better, as it is not so readily affected by water getting under the blocks. The author's statement that with a 16-lb. treatment there was absolutely no water absorption, requires further explanation. Under ordinary conditions there is usually some drying out and some water absorption, with contraction, swelling, and expansion of the wood block. This depends on the kind, quality, size, closeness of grain, extent and method of seasoning, etc., of the timber, on the temperature, pressure, duration and method of treatment, and the character of the preservative used; also on the method of pavement construction, the joint filler, the amount of traffic, the grade, the exposure to the sun or to moisture, and to various other local conditions. Wood block is like sheet asphalt in that it is better under sufficient traffic to iron out and seal the wearing surface. Mr. Thomes.

The specifications prescribing the number of annual growth rings per inch for long-leaf yellow pine is a step in the right direction. Only those who have had actual experience can realize the difficulties of grading Southern yellow pine satisfactorily to all parties. When sawed into lumber it seems to be practically impossible to distinguish some slow-growth short-leaf and other pines from some quick-growth long-leaf pine, or to distinguish second-growth timber positively, and the most feasible method of determining their classification appears to be by the average number of annual growth rings per inch. This matter is not yet sufficiently developed in the present commercial practice, but it will probably be adopted in due time. There are a number of details to be taken into consideration, and it will require time to work out the necessary practicable rules. The manufacture and treatment of creosoted wood blocks should be carefully inspected at the plant. It is desirable that the timber be inspected in the stick before it is sawed into blocks.

There is considerable difference of opinion as to the timber, the proper specific gravity, and the character and amount of wood preserving oil, the methods of seasoning and treating, and also the tests of the treated wood block. These matters require further investigation.

There seems to be no advantage in the common Western practice of laying the wood blocks at an angle of from 20° to 45° with the curb, and with several longitudinal courses of blocks along the curb. It would seem that equal if not better satisfaction is obtained with all the blocks at right angles to the curb, and with an expansion joint along the curb. Ordinarily, no transverse expansion joints are advisable.

Mr. Thomes. With all the block pavements, better results seem to be obtained with all the courses at right angles to the curb, except at intersections or along car tracks. With sheet asphalt two or more courses of brick, wood, asphalt block, sandstone or granite blocks are usually laid parallel to the rail. The blocks should be laid in a mortar bed. The joint filler will be governed by local conditions.

A matter which deserves the concerted action of city engineers and paving contractors is the standardization and adoption of uniform specifications and tests for stone block pavements, similar to those adopted for brick. If two or three uniform standard specifications for stone block pavement could be adopted, they would meet the requirements for practically all conditions. The stone quarries could work their plants to better advantage, and the cost of the blocks would be reduced. The stock blocks could be kept on hand, and their delivery facilitated. At present there is such a diversity in the various stone block specifications that it is impracticable for a quarry to keep sufficient stock on hand, and the construction work is often delayed in obtaining the proper blocks. The different specifications have limits for the depth of block varying between 4 and 8 in. As a depth of from 3 to 4 in. has been found to be sufficient for brick, wood, and asphalt block, a depth of 5 in. should be sufficient for stone block, even under the heaviest traffic, because the pavement will probably fail on account of the unevenness of the surface, due to the non-uniformity of the stone and the concentration of traffic, or because of poor foundation, before it will fail on account of lack of depth of the stone blocks. The character of the stone block will depend on traffic and other conditions. If more attention is paid to securing a good quality of stone block pavement with a smooth surface and close joints, it will wear much longer and more uniformly. Although, with this kind of pavement, there will be an increased cost, due to the better quality, dressing, and laying of the stone block, there will be a decreased cost in transportation and excavation, and closer joints can be secured more easily with a shallower block. Considering the decreased cost of maintenance, ease of traction, saving in wear and tear of vehicles, diminished noise, improved general appearance, and other advantages, a good stone block pavement will be cheapest in the long run.

By co-operation among the various engineers and contractors to secure the essentials without unnecessary refinements, pavements can be improved without much increase in cost. While it is advisable to improve the general condition of stone block pavements, care should be taken not to go to extremes. The new granite block specifications of Newark, N. J., contain a number of improvements, but go to uneconomical extremes, for instance:

"The blocks must have nearly plane surfaces for all of the faces of each block so that when a straight-edge of the full length of the block

Mr. Thomes.

is applied in any direction upon said faces, there will be no depressions or bunches which shall deviate more than $\frac{3}{16}$ of an inch from said straight-edge."

Much of the new stone block pavement in Chicago consists of a reddish Wisconsin granite, which has a smoother head and seems to be of a better quality than most of the Eastern granites.

While actual long-time service is the best test of any stone block, physical and chemical tests would be of great advantage, if proper tests can be evolved, which will give fairly reliable indications of the actual wearing qualities on the street. Considerable work along these lines has been done by the United States Office of Public Roads. Further information relative to actual wear and to tests on stone paving blocks would be of interest, also any experience with rolling stone block pavements. It was stated in Cleveland that the Medina sandstone block pavements were rolled in the same manner as the ordinary brick pavements. This appears to have some advantages.

A grout joint filler for stone blocks is preferred where the traffic can be diverted. A bituminous filler is used where the street cannot be kept closed or where the pavement is likely to be opened up frequently. A sand joint filler is also used. The grout varies from 1 part of cement to from 1 to 4 parts of sand.

The asphalt block pavement is worthy of further investigation by municipalities. Some of the early asphalt block pavements proved defective; but the present improved asphalt block, which is carefully manufactured under a pressure of 6 000 lb. per sq. in., is extensively used in New York City, and, on the whole, it is giving good satisfaction. It is somewhat more expensive than sheet asphalt, but is less slippery and is especially suitable for steeper grades and for places at a distance from an asphalt plant, as it can be laid and repaired easily. The asphalt blocks are 12 in. long, 5 in. wide, and 3 in. deep. They are laid upon a concrete foundation in a $\frac{1}{2}$-in. mortar bed, with close joints filled with sand. The asphalt blocks are composed of asphaltic cement, crushed trap-rock passing a $\frac{1}{4}$-in. screen, and limestone dust. The block has a specific gravity greater than 2.5.

A cement concrete wearing surface does not appear to meet with much favor in the larger cities. It is suitable for alleys and streets with light traffic, but it is questionable whether it will be satisfactory under heavy traffic in the large Northern cities. It has some advantages, and, with further improvements, it may prove satisfactory in many cases.

Kentucky rock asphalt pavement has met with success in some places and with failure in others; it is difficult to obtain uniform material, or bitumen of the proper amount and character, and it wears unevenly in many cases. Other bituminous pavements have given better satisfaction.

Mr. Thomes. The present sheet asphalt specifications and methods of supervision in Chicago rank with those of any city in the United States. The method of control of asphalt construction is similar to that of Washington, with some modifications. This matter, as well as so many others, is described so fully by Mr. Green that the speaker need not refer to it.

The parks and the boulevards connecting the parks in Chicago are under the supervision of three commissions appointed by the State, and they are somewhat independent of the city officials. The South Park Commission has made extensive experiments with bituminous road materials, and appears to be getting very good results with a fine asphaltic concrete mixed in a specially-designed portable asphalt plant and laid by its own force. This so-called "mineral rubber" pavement, which is similar in some respects to the bitulithic pavement, is laid either on a concrete or a finished macadam foundation. It has not been down long enough to demonstrate its durability. Considerable asphaltic concrete wearing surface is also being laid by the municipal asphalt plant at Pittsburg. In Buffalo and in Rochester fine stone which will pass a screen of $\frac{3}{8}$-in. mesh and be retained on a $\frac{1}{8}$-in. screen has been incorporated into the regular asphalt sand wearing surface, to the extent of about 20 per cent. This has been in use in Buffalo for four years, and it is proving satisfactory. With the use of fine stone the wearing surface is not so slippery, and it does not wave and creep as much as the ordinary sheet asphalt, but it requires closer attention in raking and rolling to secure a smooth uniform surface.

In Rochester, it was stated that almost all the sheet-asphalt binder stone was coated with coal-tar and also that the wearing surface along the gutters was painted with coal-tar instead of asphaltic cement. This is not the customary practice, and it is a point which merits further discussion. The asphalt and other pavements in Rochester are in very good condition.

Considerable bituminous macadam work is being done by the various State highway departments.

There is such a diversity in the paving practice of different cities that it is difficult to compare the various pavements unless one has complete detailed information on all conditions affecting the work, including records of tests, construction, traffic, wear, etc. What would be considered heavy traffic in one city would be light traffic in another. Steps should be taken to enlist the co-operation of the various cities, as far as practicable, in adopting uniform methods and records of paving work, so that the results might be comparable and of mutual benefit to all.

Mr. Lamb. RICHARD LAMB, M. AM. SOC. C. E.—Attention is called to a few important considerations in connection with that part of Mr. Green's paper which treats of wood block pavements.

Mr. Lamb.

No specifications issued by engineers show less digested thought than those for wood block pavement. This, however, is quite natural, as the basis of the subject is essentially chemical and not engineering, and the promotion of the business has not been in the hands of engineers, but of manufacturers, whose aim, as a rule, seems to have been to advocate specifications which would insure a monopoly for their particular organization rather than the ultimate best interests of the wood block paving business.

Engineers have been led to specify that dead oil of tar obtained from coal only shall be used, and to specify in the next paragraph, that 25% of resin—a product of the pine tree, and not of coal—shall be put into the heated creosote oil and be injected into the wood blocks. Further, they have been led to demand that specimens of the oil, the resin, and the resin and oil mixed, be submitted with bids for their work, as though they were not aware that what they were calling for was not resin, but pitch.

It is generally known that some years ago wood block pavement treated with pitch, was laid in New York City and proved a failure. The treatment of wood with resinous vapors was tried by Lukin, at Woolwich, in 1812, and proved an absolute failure. The Haskin Vulcanizing process solidified the resin in heart pine, but was of little value. The plant, which was built in New York City, was used subsequently for the creo-resinate process, which combines the use of resin with the commonly accepted wood preservative, creosote oil.

Mr. Green hints that, in Chicago, wood block paving promoters have been trying to dictate the specifications. He states: "It was early recognized that to use as a standard the treatment advocated by any one company was anything but scientific."

Recently the specifications for paving in a nearby city were brought to the speaker's attention. The engineer, after specifying the creo-resinate process, demanded the submission of samples of wood blocks taken from some street in New York City in which they had been laid for five years, stating that: "It is not the intention of these specifications to permit experimenting." That engineer was an honorable man, but his error lay in not having permitted the specimen blocks to be taken from other cities where they had been in use successfully for five years, as only one company manufacturing wood blocks could meet this feature of the specification, and hence that company could control the contract. It is evident that this engineer would welcome a standard specification which would eliminate, as far as possible, the experimental features of wood block pavements. To his credit, however, he inserted a clause stating that if a bidder proposed to use blocks other than those specified, a sworn affidavit describing the materials and samples of the same, must be submitted with the bid. The bids were $3.16, $3.52, and $3.56 per sq. yd., respectively. The last two bids

Mr. Lamb. were submitted by companies affiliated with the manufacturers of the blocks laid in New York City five years ago. The low bid was for standard creosoted blocks.

Any of the well-established creosoting companies can put resin in their creosote oil, and they can use creosote containing the heavier tar, but engineers cannot get them to furnish blocks other than those treated with standard or regular dead oil of coal-tar, which has passed the experimental stage and has attained results showing a life of 40 years or more.

In 1880 the Society appointed a committee to report on the best preservative for wood. After a long and careful investigation, the committee, in 1885, unqualifiedly reported in favor of standard dead oil of coal-tar.

Rotting is caused by the fungi and bacteria germinated by the alternate wetting and drying of wood. If wood is kept always wet or always dry, it will not decay for a great many years. It is a fact that 12 lb. per cu. ft. of dead oil of coal-tar has preserved wood for more than 30 years. The poisonous or antiseptic nature of the naphthalene in the dead oil of coal-tar undoubtedly had much to do with this result, as it would require more than 20 lb. per cu. ft. to saturate the pores of the wood thoroughly so as to exclude the moisture completely.

It has been argued by good engineers that the requirements for the treatment of wood blocks for paving differ from those for wood to be used for other purposes, in that the blocks must not only be preserved from decay, but must be made water-proof in order to keep them from swelling. Experience has shown that when blocks have been properly treated with 16 lb. per cu. ft. of the dead oil of coal-tar commonly used by standard companies, they do not swell more than those treated with creosote and a special water-proofing material.

The speaker knows of an extensive pavement, where 16 lb. per cu. ft. of regular creosote oil was used, and there has not been the slightest evidence of swelling, in spite of the fact that the blocks were laid very tightly.

As Mr. Green observed, Trautwine states that white pine expands 1 part in 440 000 for each degree, Fahrenheit. With the proper number of joints, there is no trouble from expansion; therefore, in treating wood blocks, the attempt should be simply to provide the preservative which has been proven by actual experience to make the wood most durable.

The special water-proofing materials penetrate the surface of the blocks only a short distance, and are not stable. When the blocks are cut for breaking joints, the interior surfaces show none of the special water-proofing, and, if not otherwise properly treated, they make an entering place for moisture.

In 1894 the speaker made some observations on the subject of wood

preservatives,* in which he contended that, in dead oil of coal-tar, naphthalene was the essential attribute for preserving the wood. He has modified his belief in this regard. Since that time the United States Forestry Department has made extensive analyses of preservatives in woods which are known to have been treated and exposed for a number of years. The results for some of the paving blocks are given in Table 3. Mr. Lamb.

TABLE 3.

Average service.	Creosote left, in pounds per cubic foot.	Distillation of Extracted Oil:							
		To 205°	205° to 245°	245° to 270°	270° to 320°	320° to 420°	Residue above 420°	Solid naphthalene.	Solid anthracene.
23.6 years.....	15.70	29%	21.34%	21.39%	41.74%	11.25%	18.64%	12.52%	40.40%
9 years poor service	5.77	.....	14.41%	19.27%	27.68%	19.03%	9.93%	.....	.....

This would indicate that the effective preservative is most largely naphthalene and anthracene, and not the tar acids or the heavier distillates which go over at a higher temperature than 400° cent. The latter constituents are mostly tar, and have a specific gravity of about 1.2.

The specific gravity of the dead oil of coal-tar used by standard creosoting companies is not less than 1.03, or more than 1.10. If a much heavier oil than 1.10 is used, great heat in the cylinders is necessary to liquefy it, and keep it liquefied, so that it can be injected into the wood.

The speaker has built a number of kilns for drying lumber, and has observed that when the heat is greater than 260° Fahr. the wood is cooked and made brittle, the cells are broken down, and the wood is injured. If the strength and life of the wood are to be preserved, those attributes of dead oil of coal-tar which have to be heated excessively in order to inject them into the wood, cannot be used.

Captain John C. Oakes, U. S. A., in a paper entitled "Difficulties in Practical Work of Creosoting Timber," states as follows:

"An oil having a specific gravity as a whole of more than 1.07, should be looked on with suspicion and investigated. Unless it is almost wholly lacking in the low-boiling fractions, it possibly has some hard pitch or undistilled coal-tar mixed with it."

A number of Government engineers specify that not less than 40% of the oil shall be naphthalene, and, at one time, the speaker was of

* *Transactions*, Am. Soc. C. E., Vol. XXXI, p. 238.

Mr. Lamb. this opinion. A large consignment of creosoted piles for use in the construction of a Government railroad trestle, was rejected by the Government engineer, because the oil contained 45% of naphthalene. The specifications read that not less than 40% of naphthalene should be in the oil. The engineer argued that he knew nothing about this substance, and that if his report showed 45%, the Washington authorities might blame him for accepting piles having too much naphthalene in them. The speaker laid the claims of the authorities on wood preservation before the Chief of Engineers, U. S. A., showing that naphthalene was the essential constituent of oil for preserving wood against rotting and the terèdo. The Chief Engineer telegraphed to his subordinate at once to use the piles, and expressed surprise that the work had been held up for so obvious an error in the reading of the specifications.

The speaker has since concluded that this subordinate engineer was more nearly correct in his conclusions, as far as the value of the oil is concerned, than even he himself knew. If too large a percentage of naphthalene is specified, it is almost certain that a large percentage of the tar acids or lighter volatile attributes will accompany the oil. These soon leave the wood.

If the degree of heat specified for distilling is greater than the minimum necessary to extract all the lighter oils and some of the naphthalene, the volatile oils will be excluded, and if the maximum degree of heat be specified, which will fall below that necessary to take over the heavy tars, the oil will contain most of the naphthalene and nearly all, if not all, the anthracene. These degrees of heat should be positively determined, and then incorporated in specifications.

Captain Oakes stated:

"If the creosote is obtained from works where the carbolic acid is extracted, the oil will have a low percentage of tar acids. If naphthalene has been extracted, the oil will be low in that quality, and if roofing pitch has been manufactured, there will be almost no anthracene, because the distillation ceased before the oils containing anthracene were distilled over."

From these facts it may be seen that useful information as to the probable value of any creosoting oil can be obtained in advance of an analysis, by ascertaining the kinds of materials produced by the manufacturers who are to furnish the oil.

A consideration not less important in wood block paving is the kind of wood to be used. After using many kinds of hard woods in Europe, engineers have found from experience that Baltic pine, *Pinus sylvestris,* is the best and most durable. This wood corresponds to the American *Pinus mitis,* or shoft-leaf sap pine; but it is not like the *Pinus rigida,* or old field pine, which is also short-leafed; the latter is very coarse-grained.

Mr. Lamb.

In the East it has become common practice of late, to call for *Pinus palustris* or long-leaf or heart pine. It is questionable whether this is best. Short-leaf pine is the stronger, although of shorter life than the long-leaf pine. However, when short-leaf pine is creosoted, it lasts as long as the long-leaf.

While the short-leaf pine has less resilience, it will stand the wear of wheels longer than long-leaf pine, on the same principle that the best bearing metals are not the hardest.

It is likely that tupelo or black gum will come into popular favor. There is a vast quantity of this wood in those districts of the United States where creosoting works are located. At present it is scarcely used for any purpose. The negroes say that black gum is so mean that even the lightning will not have anything to do with it, as it is commonly believed that lightning never strikes black gum trees. The wood is hard to split; it has little fuel value, and has a bad feature of warping and twisting. However, the speaker has manufactured a large quantity of it into long slender poles which were required to be straight and would be ruined by warping or twisting. The method was as follows: The trees were girdled when the sap was down. The sap rises twice a year, in March and in August. After letting the girdled tree remain for one year, it was then ready to be made into timber or lumber, and would keep its shape. It was also in better condition to creosote, and the speaker believes that the wood will wear better than any that could be obtained at the same cost.

The object of these remarks is to show, as briefly as possible, some of the considerations that make it important for the Society to appoint a committee to investigate the subject thoroughly, and to report on the best complete specifications for wood block paving.

MEMOIRS OF DECEASED MEMBERS.

NOTE.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

WILLIAM CHARLES KERNOT, M. Am. Soc. C. E.*

DIED MARCH 14TH, 1909.

William Charles Kernot was born at Rochford, Essex, England, on June 16th, 1845. With his father, he went to Australia in 1851—the year in which gold was discovered in large quantities in Victoria.

He received his primary education at the Geelong National Grammar School and, in 1861, matriculated at Melbourne University, where he passed with distinction through the Arts and, subsequently, the Engineering Courses, his name appearing seven times on the honors list, three exhibitions and one scholarship being also awarded to him.

Professor Kernot began his engineering practice on the Geelong Water-Works, and this was followed by work on the Coliban Water-Works and in the Railways and Mines Departments.

In 1869, he began his connection—in the capacity of lecturer—with the staff of Melbourne University. In 1883 he was appointed Professor of Engineering, a position which he filled, and with which he was identified, until his death. In 1888 he also fulfilled the functions appertaining to the Chair of Natural Philosophy.

Professor Kernot served six times as President of the Victorian Institute of Engineers, in 1886, 1890, 1897, 1898, 1906, and 1907. He was also a Member of the Institution of Civil Engineers, London, and of the Victorian Institute of Surveyors. He was a Vice-President of the Royal Geographical Society of Australasia, a Fellow of the Royal Geographical Society of England, for several years President of the Royal Society of Victoria, and a Past-President of the Engineering Section of the Australasian Association for the Advancement of Science.

Among the numerous public and advisatory positions filled by Professor Kernot at various times may be cited the following: Member of the Municipal Surveyors Board for forty years—serving thirty years as Chairman of that board; Member of the old Board of Public Health; Member of the 1880-1881 Melbourne Exhibition Committee; and Chairman of Juries for the Melbourne Jubilee Exhibition, 1888. He was a Member of Council and one of the Trustees of the Melbourne Working Men's College, of which school he was also President from 1889 to 1899. He was also a Member of the following Boards:

* Memoir prepared by E. Manchester, Assoc. M. Am. Soc. C. E.

Victorian Railway Enquiries and Commissions; New South Wales Bridge Commission; Royal Commission on the Stability of Victorian Railway Bridges; Board of Visitors to the Melbourne Observatory; Yarra (River) Floods Board; Telegraph and Telephone Tunnel Board (1886); Melbourne Drainage Commission (1889); Board of Examiners for Water Supply Engineers; Directorate of the first Australian Electric Light Company; Trustee of the Royal Park, and numerous other offices. Professor Kernot made three trips to Europe and America, and was in South Africa during the progress of the Boer trouble.

He made many donations to public purposes, among others being liberal benefactions to the Working Men's College, also the sum of £1 000 to found a Scholarship in Metallurgy, and a sum of £2 200 to establish a Scholarship in Chemistry and Natural Philosophy, at Melbourne University.

Without doubt Professor Kernot was one of the kindliest and most genial of men, at all times absolutely unsparing of himself in the interests of others—one of Nature's own noblemen—and his death has caused a gap in the life of Victoria, which will be hard to fill.

In public life it would be difficult to say with what engineering function, with what important engineering movement in Victoria for more than forty years, he was not identified, but his chief work—his life's work—was associated with his University duties. His students received more than technical training, for Professor Kernot's genial kindly nature invited confidence, and his advice left a lifelong impression upon them. Above all he was so absolutely an upright man, that every day the example he set of what an honorable member of an honorable profession should strive to attain was above price to his students.

Professor Kernot was unmarried. He was elected a Member of the American Society of Civil Engineers on March 6th, 1889.

HENRY BROWN RICHARDSON, M. Am. Soc. C. E.*

DIED AUGUST 21ST, 1909.

Henry Brown Richardson was born on August 23d, 1837, in Winthrop, Me. He was the son of the Reverend Henry Richardson and Eunice Farley Thurston. Among his ancestors and relations were many historic New England names. An adventurous and healthy boyhood, passed along the valley of the Androscoggin River and among the foothills of the White Mountains, laid the foundation for his understanding of Nature's operations, and gave him the bodily strength and endurance which subsequently formed part of his professional equipment.

Having acquired the education which the local schools extended, he left home for broader fields of work and life than the neighborhood afforded. His tastes and acquirements led him to seek employment of a technical and constructive character, which he first found in engineering offices in Portland, Me., and afterward, as his wanderings extended, in Providence, Boston, Milwaukee, Chicago, and other Western cities. In the intervals in such employment, he taught school in Illinois and elsewhere.

In 1860 Mr. Richardson reached Louisiana, where he was destined to find his future home and life work. He readily secured employment with the railroads and other industrial developments in that region, and particularly under the State authorities then in charge of the levee system of the Mississippi River. These engagements were soon interrupted by the outbreak of the Civil War.

In April, 1861, Mr. Richardson enlisted in an infantry company which was promptly ordered to Virginia. He was then immediately detailed on engineering duty, and served successively on the staffs of Generals Dick Taylor, Early, and Ewell. His services, particularly in rapid, daring, and accurate reconnaissance, were distinguished and led to his steady promotion until, by the time the war was half over, he had attained the rank of Major of Engineers. He was desperately wounded at Sharpsburg and Gettysburg, and, from the latter field, was carried to Johnson's Island where he remained a prisoner until almost the close of the war.

Immediately on the restoration of peace Mr. Richardson returned to Louisiana and resumed his engineering practice, of which the reclamation of the Mississippi Valley from overflow formed a large and important part, and, becoming his life work, claimed more and more of his service.

In 1877, on the reorganization of the State levee system, he was appointed, with B. M. Harrod, Past-President, Am. Soc. C. E., and the

* Memoir prepared by B. M. Harrod, Past-President, Am. Soc. C. E.

late T. S. Hardee, M. Am. Soc. C. E., to form a Board of Engineers having control of the levees of Louisiana. In 1881 he was made Chief Engineer of this Board, which position he retained for twenty-three years, until 1904. In this office he was charged with the improvement and maintenance of 1 500 miles of levee containing 150 000 000 cu. yd. of embankment, and protecting 15 000 sq. miles of territory from overflow, at an average annual expenditure of more than $1 000 000.

Mr. Richardson's duties as State Engineer brought him in constant and close relations with the conflicting interests of the work, the riparian owners, and the numerous contractors, but were discharged without friction and in a manner which inspired all concerned with the most absolute confidence in his knowledge, judgment, and fairness. The evidence of the wisdom of his administration and the confidence with which he inspired all with whom he dealt is that he retained an office which was subject to change at every change of State administration for twenty-four years, until he resigned it to accept service under the United States as a Member of the Mississippi River Commission.

His mental vigor and readiness, great physical endurance and resolution, equipped him well for a branch of professional work which is familiar with dangers, exposure, and emergencies, and where for months great interests hang in jeopardy from floods. His strong and active powers of observation, trained by years of experience, gave him great authority as a student of alluvial streams, and enabled him to render probably the most eminent service of any of his colleagues in the great work of reclaiming the Mississippi Valley from overflow.

In 1904 Mr. Richardson resigned as Chief Engineer of Louisiana to accept membership on the Mississippi River Commission, for which it was universally conceded that his special experience eminently fitted him.

With Majors Crittenden and Dabney, he was a member of the Board to report on the "Rectification of the Sacramento and San Joaquin Rivers and their principal tributaries." He was also on the Board for the Drainage of New Orleans, and was in conference on many other engineering projects of magnitude.

While this memoir is chiefly concerned with Mr. Richardson's career as a Civil Engineer, it would give but a meager idea of his character and influence if no mention were made of the personal traits for which he was beloved by all who had the privilege of his intimacy, which earned him the trust and respect of the community.

Mr. Richardson's character was one of peculiar modesty and reserve, combined with unusual firmness and courage. He was patient and unselfish to a degree. He was incapable of engaging in any struggle for personal distinction or advantage, and shrank from any prominence which was not based on the merit of his work. He asked

no favor, and all his work and fame came to him as the result of successful achievement and established character.

In 1867, Mr. Richardson married Miss Anna Howard Farrar, of Tensas Parish, La., and the union proved to be one of great happiness. There were born to them nine children, of whom all but the eldest survive. Of six boys, one is an Associate Member of this Society, and three others are in engineering employment.

Mr. Richardson was elected a Member of the American Society of Civil Engineers on May 7th, 1879, and served as a Member of the Board of Direction from 1900 to 1902.

Vol. XXXV. NOVEMBER, 1909. No. 9.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

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THE NEW YORK TUNNEL EXTENSION OF THE PENNSYLVANIA RAILROAD. CONTRACTORS' PLANT FOR EAST RIVER TUNNELS.

By Henry Japp, M. Am. Soc. C. E.

To be Presented December 15th, 1909.

The minimum plant to be provided by the contractors for this undertaking was specified by the Railroad Company as follows:

"The tunnels shall be driven eastward from shafts in Manhattan and westward from the temporary shaft to be built near East Avenue in Long Island City, making a total of eight headings, in all of which work shall be prosecuted simultaneously with the utmost diligence practicable. The Contractor must provide on each side of the river an adequate plant, including boilers, air compressors, hydraulic machinery, dynamos, and all other necessary plant, with a reasonable duplication to meet unusual and unexpected emergencies.

"The air compressors shall be of sufficient capacity to deliver regularly into each heading at least 300 000 cu. ft. of free air per hour at a pressure of 50 lb. per sq. in. above the normal air pressure, and for a larger amount if found necessary during the progress of the work. The air for the compressors must be drawn from the exterior of the power-house, and the intake located so as to give pure air. The air shall be cooled, and oil and other impurities removed as completely as practicable before delivering it into the headings.

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

"In order to provide a reasonable margin for repairs and contingencies, a spare compressor and boiler plant shall be provided on each side of East River and kept in good condition and repair, ready for immediate use. The capacity of the spare plant shall be 25% of that required in the preceding paragraph for regular operation.

"Provision must be made for storing in tanks at each boiler-house enough feed water for 12 hours' supply, unless connections can be made with two independent and sufficient sources of supply.

"The air shall be delivered into each heading through two supply pipes of such capacity that the velocity of air through them in regular working shall not exceed 40 ft. per sec. These pipes shall be tapped with regulating valves in each intermediate air chamber in tunnels, and sufficient air admitted to ventilate it and to maintain the pressure required. If required by the Engineer, air shall be delivered at the shield in each section or compartment thereof where men are employed, and withdrawn therefrom in special exhaust pipes, with suitable regulating valves.

"Each supply pipe shall be furnished with two tees with valves and an intermediate valve in the supply pipe at some convenient place between the compressors and the shaft to enable a by-pass to be formed.

"A foul-air vent pipe, 6 in. in diameter, shall be carried back from the shield through each lock bulkhead to the ordinary atmosphere to ventilate the heading, and shall be provided with a 10-in. pressure-regulating valve near the shield to maintain the pressure required; the valve shall be placed so as not to be readily tampered with.

"Effective means shall be used to secure proper ventilation. The amount of carbonic acid at any working face or in any chamber must never exceed one part in one thousand parts of air, and compressors must be run so as to maintain at all times a change of air through the pressure-regulating valves. Suitable devices shall be used to deaden the noise, of the air introduced and exhausted, as much as practicable. When blasting is resorted to, special means must be provided for the rapid removal of the fumes produced.

"Bulkheads shall be built in each tunnel at intervals not exceeding 1 000 ft., and there shall at no time be an interval of more than 1 000 ft. between a shield and the bulkhead nearest to it. They shall be of concrete or of brick set in Portland cement mortar, or other plan approved by the Engineer. Each bulkhead shall be provided with two air-locks near the bottom, at least 6 ft. in diameter and 20 ft. long, for the passage of men and materials, one near the roof as an emergency lock for the passage of men only, and a pipe-lock 12 in. in diameter and 31 ft. long, with a gate-valve at each end, for passing pipes and rails. The emergency lock shall be of ample dimensions to contain the entire force employed at any time at the heading. Stairways and galleries shall always be maintained to give convenient

access thereto. All parts of bulkheads and air-locks must be of sufficient strength to sustain safely a pressure of 55 lb. per sq. in.

"The pipes necessary for air supply, ventilation, hydraulic and electric transmission, and other purposes shall be built into the bulkhead and provided with suitable connections. All pipes shall be standard, lap-welded.

"When a shield has been driven 500 ft. or more from the shaft, at least two bulkheads shall always be in use if compressed air is used.

"A safety screen, extending from the roof downward into the tunnel, of design approved by the Engineer, shall be maintained within 100 ft. of each working face, and others shall be built at intermediate points between the working face and the nearest bulkhead, if necessary, to maintain a chamber filled with compressed air along the tunnel roof and give access to the emergency lock. The galleries shall extend from the safety screen nearest the working face to the first bulkhead.

"The shields must be of ample strength and of the best materials, must be provided with hydraulic rams of sufficient power to move them along the alignment laid down on the plans and profiles, and must have adequate arrangements for the rapid execution of the work and for the safety of the men employed therein. The Contractor will be required to make use of the most effective devices in the construction and operation of the shields.

"Detailed plans and specifications of the shields, hydraulic presses, fittings, and other appliances must be submitted to the Chief Engineer before the construction of the shields is begun, and must be modified or amended, if required by him, and then built in exact accordance with the plans in every respect. Such submission of plans to the Chief Engineer shall in no way relieve the Contractor from his responsibility for carrying out the work embraced in the contract."

These specifications were a great help to the contractors in bidding, and also in deciding what plant should be installed.

The original plant fulfilled these requirements, but the porous material overlying the tunnels needed more air than specified, and the plant was increased.

Air Compressors.—In endeavoring to select the best air compressor for continuous night and day service, plants erected by the various manufacturers, where available, were examined, and indicator cards of the steam and air cylinders were taken, and, where plants were located too far away, indicator cards were submitted for inspection.

After studying their relative merits, it was decided to adopt the Ingersoll-Rand air-moved valve, on account of the larger area and free opening for inlet and discharge, and the reduced clearance spaces.

FIG. 1.—AIR COMPRESSORS, MANHATTAN PLANT.

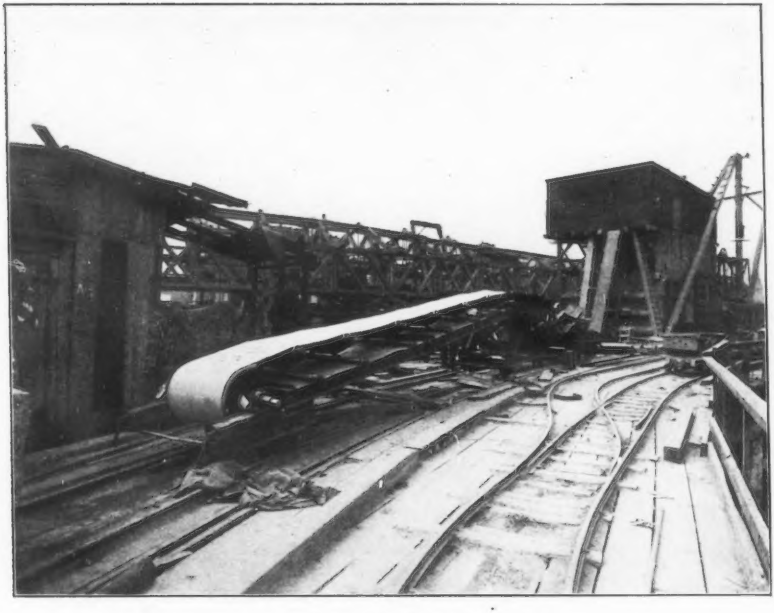


FIG. 2.—BELT CONVEYOR.

This was chosen in preference to other types of inlets, with poppet discharge valves, as a high piston speed was necessary on account of the limited area for plant installation at the disposal of the contractor.

This choice was amply justified, especially when the manufacturers replaced the original cast-steel valves by oil-treated steel forgings, which were very successful, and never had to be replaced during four years' operation.

One objection, in addition to the usual ones advanced, which influenced the decision against Corliss inlet valves was that, in compressors using such valves, the point of opening or inlet is fixed, whereas it ought to vary with the final discharge pressure, and as the pressure required in the tunnels and caissons called for engine-room pressures varying from 50 lb. per sq. in. downward, a fixed point of inlet was not desirable.

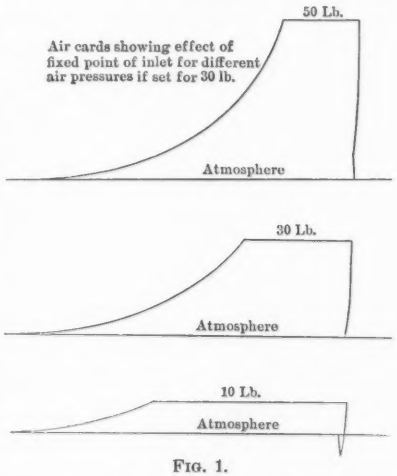


FIG. 1.

The air which is compressed in the clearance space expands to atmospheric pressure before a poppet valve will lift, but, with Corliss valves, if the inlet is set at a point in the stroke before the clearance air expands to atmospheric pressure, there is a puff of air outward, and, if the Corliss inlet valve does not open until after the compressed air in the clearance space has expanded below atmosphere, there will be a sudden inrush of air. In both cases energy is lost.

In Fig. 1 is shown a typical air card, with the inlet opening set at the correct position for a 30-lb. final pressure, and the effect of maintaining it at the same point, if the final pressure should be raised to 50 lb. or reduced to 10 lb. This is well illustrated on cards taken from the electrically-driven Corliss inlet compressors shown later. In most high-speed air compressors fitted with mushroom or poppet valves, a small loop is evident on the air cards, because a slight vacuum must be present before the inlet valve is drawn inward. With careful adjustment, the air-moved valves can be set so that there is no loop at a

certain speed, but if the engine runs at a different speed a slight loop occurs, as will be shown later on actual air cards.

In determining the size of units to be selected for the low-pressure air work, on laying out alternative plants based on different sizes of units, it was found that the most convenient size for the site available was a 5 000-cu. ft. per min. machine. The duplex compound Corliss condensing type — supplied by the Ingersoll-Rand Company — was selected, with steam cylinders 16 and 34 in. in diameter, and two air cylinders 26½ in. in diameter with a 42-in. stroke. The steam valves are of the Corliss release type with vacuum dash-pots. The air-valves are of the positive, pneumatically-operated type (Fig. 2), with two intake valves, and two discharge valves at each end of the air cylinder.

Each valve is 7 in. in diameter, giving a clear opening of 13.9% of the cylinder area at each end, which gives the air a speed of 7¼ times that of the piston provided the valve lifts one-quarter of its diameter from the seat. At 100 rev. per min. the piston speed was 700 ft. per min., and the speed of the air through the valves was 5 000 ft. per min. It was found that, in order to maintain perfect indicator cards, the valves need not lift the full quarter diameter.

They are all direct-lift valves, each with a central stem coming out through the cylinder head and carrying a piston which works in a small cylinder mounted on the head. These cylinders or dash-pots have the double function of operating the valves and of cushioning to prevent pounding and noise. The discharge valves are guided in the solid metal of the head and the inlet valves are guided in sleeves which are integral with the inserted valve seats, so that all the valves must seat true. The inlet valve has a spring-steel ring snapped into a recess around it, which prevents the valve from being sucked into the cylinder if the stem should break. The valves and appurtenances are interchangeable, and are easily removable.

At the side of the air cylinder is mounted a valve chest which has a flat seat and a sliding valve, the stem of which has on it a sliding sleeve connected by a rod to a horizontally-vibrating lever which receives its motion from the main piston-rod coupling. The motion of this sleeve, therefore, is coincident in time and direction with that of the main piston, but it only moves the valve rod a short distance at each end of the stroke, when it strikes adjustable collars on the stem. The ports at each end of the chest are connected to the outer ends of

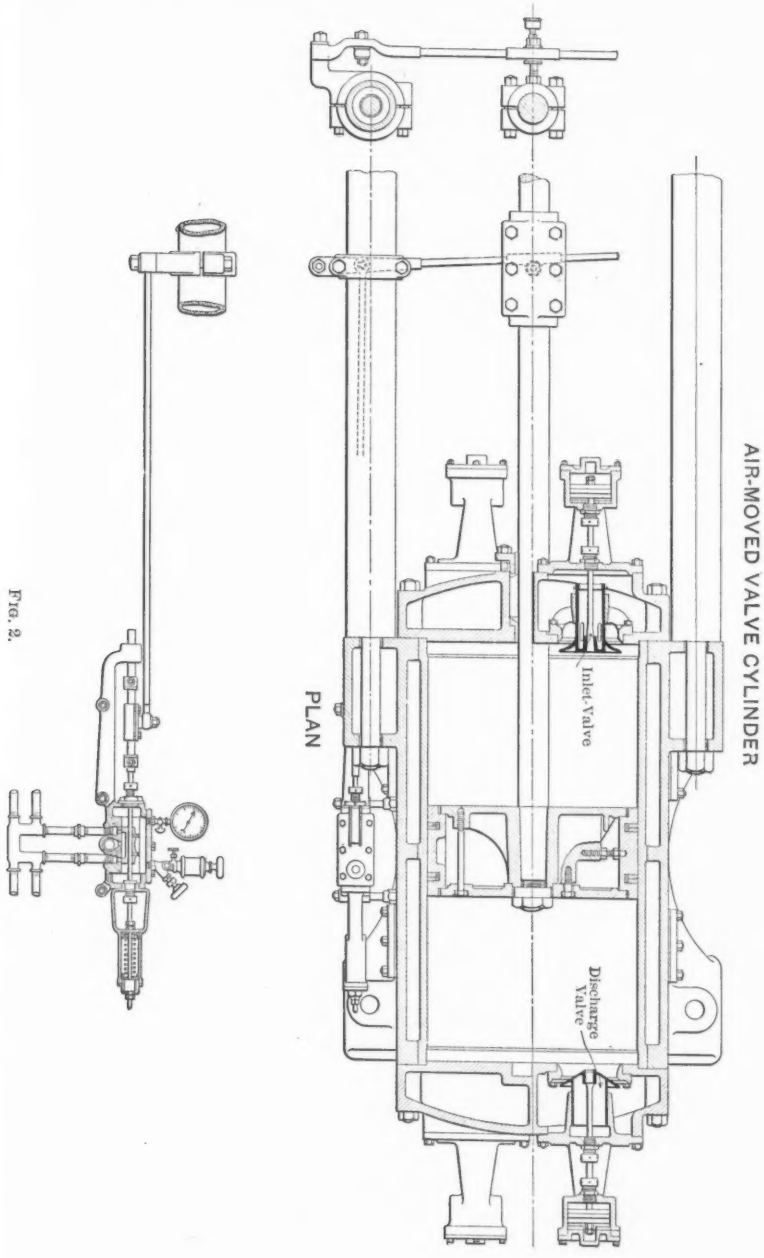


FIG. 2.

the valve-operating cylinders. No air pressure is required when starting the compressor, the inlet valves being moved both ways by the air passing through them. As the pressure accumulates when the compressor begins to work, the valves are under full control after a few strokes.

The arrangement as described applies to the low-pressure cylinder. Supposing the compressor to be in full operation and the compressor piston at the end of its stroke, as it starts away from the cylinder head, the action of the sliding valve supplies air to the outer ends of the inlet-valve cylinders and the valves are opened. As the compressor piston moves forward, the sliding valve is brought to a central position by a spring, and the air is discharged from the valve-operating cylinders without moving the valves. When the compressor piston reaches the other end of its stroke, the sliding valve is moved again, air is admitted to the other ends of the valve cylinders, and the inlet valves are closed.

Connected with the inner ends of the discharge-valve cylinders are pipes opening into the compressor cylinder, so that, as the compressing piston advances and the pressure rises, the same pressure appears in the valve-operating cylinders, tending to open the valves. This is prevented by the normal pressure which is on the backs of the discharge valves until this pressure is overcome by the combined pressure in the compressor cylinder and the valve-operating cylinders, when the valves at once fly open, and the compressed air begins to flow into the discharge passage. Each valve cylinder has a port and a pipe tapped into it, and, as soon as the piston has moved a short distance, this port is uncovered, the pressure in the valve cylinder is exhausted, and the valve is cushioned or steadied in its movement by the air in the outer end of the cylinder. As the main piston passes on, it covers the pipe opening, and, when the piston reaches the end of its stroke and the sliding valve is moved again, air is admitted to the valve-operating cylinders and the valves are free to close. The pipe from the chest is led into the main intake to prevent noise from the discharged air.

The air used for operating these valves was reduced to about one-third of the discharge pressure of the compressor, and some adjustment of the openings controlling the air supplied to these small cylinders was necessary by the operating engineers before perfectly smooth running was attained, but, when the adjustment was made, the valves dropped to their seats with practically no noise. The inlet valves re-

quired about six-tenths of 1%, and the outlet valves about three-tenths of 1%, or say, in all, 1% of the total free air compressed to operate them, if no leakage occurred.

The steam end of the air compressors was compound Corliss, with trip-gear controlled by the governor on each cylinder. In order to maintain a high economy at speeds varying from 50 to 125 rev. per min., the governor was fitted with a variable-speed device composed of two cast-iron saucers with their concave surfaces facing each other; the outer one, driven by a belt from the main shaft of the engine, engages three leather-lined rollers which in turn engage the inner saucer, sufficient pressure for friction drive being obtained by a spring on the spindle.

With the axis of the three leather-lined rollers at right angles to the axis of the saucers, the two saucers go at the same speed, but in opposite directions. If the rollers are tilted, so that they roll on a larger diameter of contact on one saucer than on the other, a different speed will result. These rollers are held in a triangular frame which does not rotate, and it is possible by a hand-wheel to alter the inclination of the rollers so that the speed of the governor can be changed. This allows the engine to run at a variable speed, with the governor floating in its critical position.

The steam cylinders and intermediate receiver were steam-jacketed, and a Cochrane steam separator was mounted on the stop-valve. Steam was admitted at a boiler pressure of 150 lb. per sq. in., and the exhaust was carried to condensers at about 26 in. of vacuum.

Table 1 gives a fair idea of the efficiency of these low-pressure units for seven compressors on the Manhattan side. From this it would appear that the mechanical efficiency of the machine is a little better than 90%, and the volumetric efficiency about 96 per cent.

Fig. 3 shows typical steam and air cards for the low-pressure machines.

On a test made for steam consumption, the result was 14.2 lb. of steam per i.h.p. per hour, compressing up to 30 lb. per sq. in.

In order to cover the demand of the specifications for 25% spare plant, a combination machine was designed which could be used either as a high-pressure machine for rock drills, or as a low-pressure machine for tunnel air. It had the same steam end as the low-pressure engines, but was fitted with two low-pressure cylinders 22¼ in. in

diameter, and two high-pressure cylinders 15½ in. in diameter. Running as a low-pressure machine, with all four air cylinders in operation, it had a capacity of 5 000 cu. ft. per min., and, if it was desired to run it as a simple high-pressure machine, the two low-pressure cylinders could be disconnected and it could then compress 1 568 cu. ft. of free air, up to 90 lb., with atmospheric intake, and 6 899 cu. ft. to 140 lb., with an intake pressure of 50 lb. per sq. in.

TABLE 1.—RESULTS OF TESTS OF SEVEN COMPRESSORS AT THE MANHATTAN PLANT.

Engine No.	Diameters of steam cylinders, in inches.	Air Cylinders. Number.	Air Cylinders. Diameter, in inches.	Stroke, in inches.	Revolutions per minute.	Cubic feet of free air compressed per minute.	Air pressure, in pounds.	Isothermal horse power.	Adiabatic horse power.	Relation: Isothermal / Adiabatic.	Steam: Indicated horse power.	Air: Indicated horse power.	Efficiency: Isothermal horse power / Air, Indicated horse power.	Mechanical efficiency: Air, Indicated horse power / Steam, Indicated horse power.	Volumetric efficiency.
1	16 and 34	2	26½	42	70	3 569	35	278.8	339	82.0	376	332	84%	88.5%	96
2	16 and 34	2	26½	42	72	3 671	37½	298.0	367	81.5	401	361.5	83%	90.0%	96
3	16 and 34	2	26½	42	65	3 314	36	262.8	321	81.8	349	318	82.6%	91.0%	96
4	16 and 34	2	26½	42	75	3 824	36½	306.0	373	81.7	407.5	376	81.4%	92.0%	96
5	16 and 34	2	26½	42	65	3 314	37	266.8	327	81.6	351	318	84%	90.5%	96
6	16 and 34	2	26½	42	85	4 324	36	343.6	420	81.8	455	419	82%	92.0%	96
7*	16 and 34	2 / 2	15½ / 22¼	42	98	5 060	92	640.0	744	86.2	865	779	82.2%	90.0%	97.2

* Engine No. 7 was operated as a high-pressure compressor, with two 15½-in. cylinders in combination with a low-pressure machine. See Fig. 4.

Although heavily overloaded, it was possible to run these machines as independent two-stage compressors. The low-pressure cylinders, of course, were too small for the most efficient arrangement of two-stage, and would require a diameter of 26½ in., but, by admitting an additional supply of air from the tunnel compressors to the high-pressure end of the machine, it was possible to get a very large output from it.

In Table 1, Engine No. 7 is the combination machine. Further details of its performance are shown on Fig. 4, operating with a low-pressure compressor as a two-stage machine.

Each air compressor was fitted with a vertical low-pressure aftercooler, 57 in. by 14 ft. 6 in., each having 920 sq. ft. of cooling surface, fitted with tinned navy-mixture brass tubes and Tobin bronze tube plates. The air from each compressor was discharged into individual low-pressure air receivers, 4 ft. 6 in. in diameter and 12 ft. high.

STEAM AND AIR CARDS FOR LOW-PRESSURE AIR COMPRESSORS

BOILER GAUGE, 145 LB. VACUUM GAUGE, 24 IN. REVOLUTIONS PER MINUTE. 70

Air Pressure 34½ lb.

FIG. 3.

The combination machines, acting as high-pressure compressors, were also fitted with a low-pressure after-cooler which was used as an inter-cooler when the machine was two-stage, and the air was discharged into a high-pressure air receiver, 60 in. in diameter and 14 ft. high.

In addition to the steam-driven, low-pressure machines, it became necessary at the Long Island City site to purchase two low-pressure, electrically-driven machines. These were supplied by the Laidlaw-Dunn-Gordon Company. Each compressor had two air cylinders, 31 in. in diameter with 42-in. stroke, with rotative inlet valves of the Cincinnati type. These were purchased in spite of the objection to the Corliss inlet noted previously, as they were to be used toward the end of the work, when the air pressure was almost constant.

They were designed for a speed of 75 rev. per min., with a rope-drive fly-wheel, 20 ft. in diameter, weighing 20 tons, and with fourteen 2-in. ropes. Horizontal after-coolers, of 1 000 sq. ft. each, were attached, and the air was discharged into air receivers 12 ft. high and 4 ft. 6 in. in diameter. They were driven by 600-h.p. Westinghouse motors, Type *CCL*, 440 volts, 3-phase, 25 cycles, running at 300 rev. per min., with a rope sheave 5 ft. 2 in. in diameter. The motors received current from three transformers of 375 kw. each, oil-insulated, and water-cooled, receiving alternating current at 11 000 volts and transforming it down to 440 volts of 25 cycles, 3-phase.

Fig. 5 shows the efficiency of these machines from actual tests.

The great drawback to these electrically-driven air compressors is that there is no way of regulating the volume of air discharged, as the speed of the motor cannot be changed.

The usual method of operating them was to open them out on two or more tunnels requiring more than their combined capacity, and adjust the volume of air by means of one of the steam engines. They worked very well and gave no anxiety, and without them one or more of the tunnels would have had to shut down.

As stand-by high-pressure machines, when the combination machines were on low-pressure duty, two Class *HC*, Ingersoll-Rand, duplex, simple, steam, two-stage air compressors were purchased, with two steam cylinders 16 in. in diameter, and air cylinders 16¼ and 25¼ in. in diameter and 16-in. stroke, and a capacity of 1 205 cu. ft. of free air per minute for each machine, for the Manhattan site.

Fig. 4.

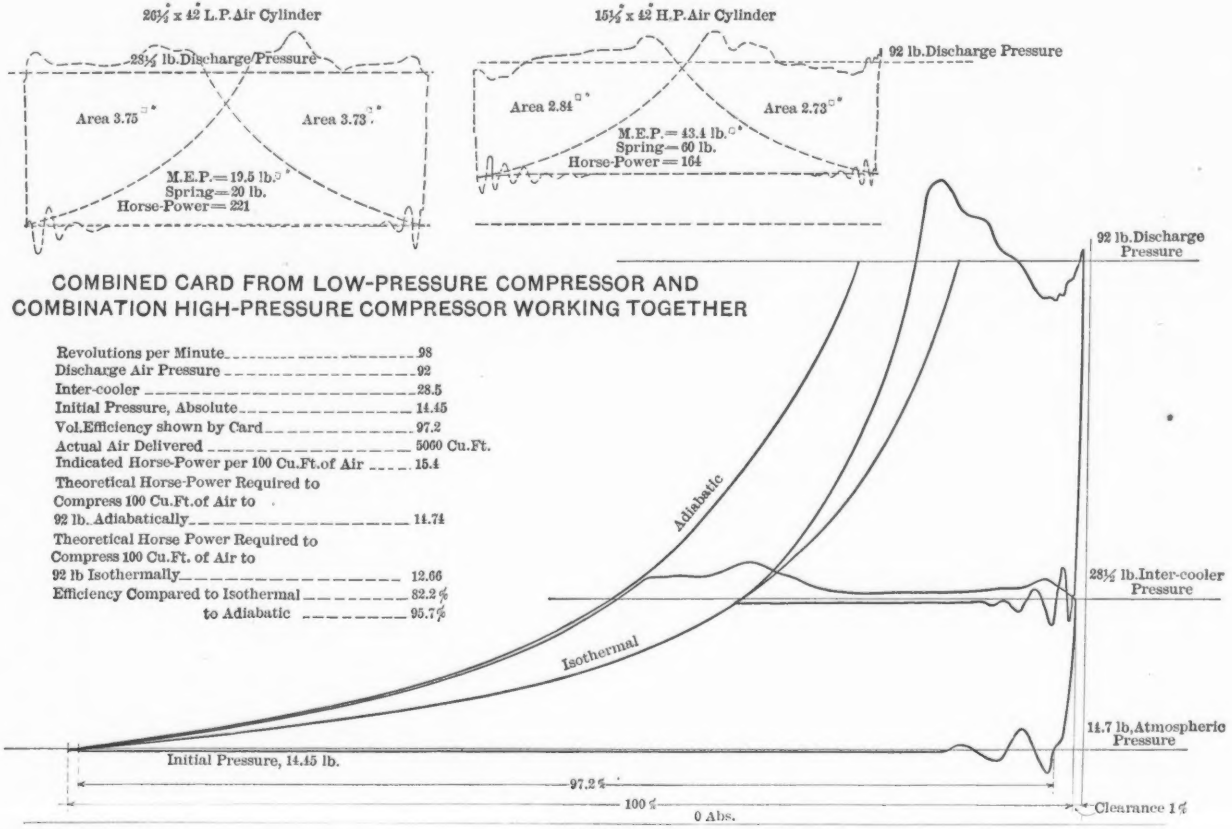
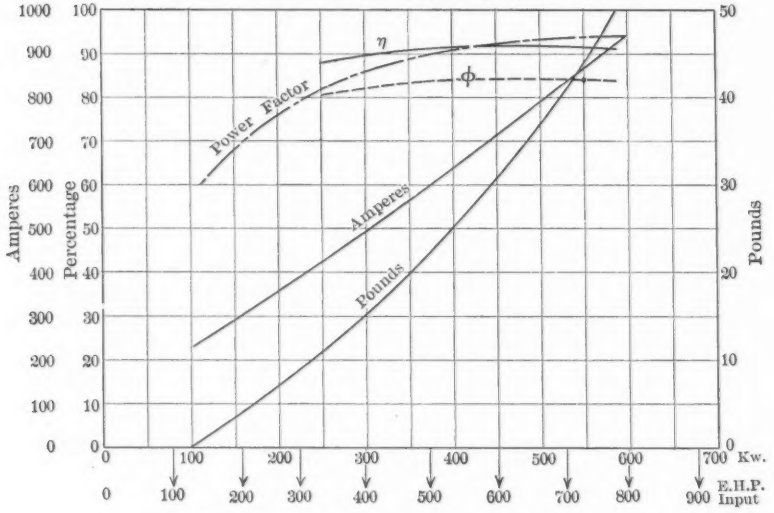

26½" x 42" L.P. Air Cylinder
28½ lb. Discharge Pressure
Area 3.75 □"
Area 3.73 □"
M.E.P. = 19.5 lb.
Spring = 20 lb.
Horse-Power = 221
15½" x 42" H.P. Air Cylinder
92 lb. Discharge Pressure
Area 2.84 □"
Area 2.73 □"
M.E.P. = 43.4 lb.
Spring = 60 lb.
Horse-Power = 164
COMBINED CARD FROM LOW-PRESSURE COMPRESSOR AND COMBINATION HIGH-PRESSURE COMPRESSOR WORKING TOGETHER
Revolutions per Minute 98
Discharge Air Pressure 92
Inter-cooler 28.5
Initial Pressure, Absolute 14.45
Vol. Efficiency shown by Card 97.2
Actual Air Delivered 5060 Cu. Ft.
Indicated Horse-Power per 100 Cu. Ft. of Air 15.4
Theoretical Horse-Power Required to Compress 100 Cu. Ft. of Air to 92 lb. Adiabatically 14.74
Theoretical Horse Power Required to Compress 100 Cu. Ft. of Air to 92 lb Isothermally 12.66
Efficiency Compared to Isothermal 82.2 %
to Adiabatic 95.7 %
92 lb. Discharge Pressure
Adiabatic
28½ lb. Inter-cooler Pressure
Isothermal
14.7 lb. Atmospheric Pressure
Initial Pressure, 14.45 lb.
97.2 %
100 %
Clearance 1 %
0 Abs.

FIG. 4.

On the Long Island City side, where not so much rock was encountered, while the combination machine was likely to be used on low-pressure duty, one Imperial Ingersoll-Rand, duplex, simple, steam, two-stage air compressor was purchased, with two steam cylinders 16 in. in diameter, and air cylinders 15 and 25 in. in diameter and 20-in. stroke, and steam and air pressure of 100 lb. per sq. in., with a capacity of 1 700 cu. ft. of free air per minute.

EFFICIENCY CURVES
FOR ELECTRICALLY-DRIVEN AIR COMPRESSORS

No.2, Laidlaw-Dunn-Gordon, Duplex, Electrically-Operated, Rope-Driven Compressor. Westinghouse Type, C.C.L. Motor, 600 h.p., Three-phase, 25 Cycles, 300 Rev. per Min. 400 Volts.



η Mechanical Efficiency $= \frac{\text{Indicated Horse Power}}{\text{Power at Motor Sheave}}$

ϕ Efficiency $= \frac{\text{Indicated Horse Power}}{\text{Electrical Horse Power}}$

Motor Efficiency 92% from one-half to Full Load
Average Volumetric Efficiency, 95.7%
Revolutions of Compressor per Minute=75
Diameter of Cylinders=31 in.; Stroke, 42 in.

FIG. 5.

For starting up the headings at the East Avenue site, two Ingersoll, straight-line air compressors were purchased, with steam cylinders 18 in. in diameter, air cylinders 18¼ in. in diameter and 2-ft. stroke. At 94 rev. per min., with a steam and air pressure of 90 lb., each had a capacity of 656 cu. ft. of free air per minute.

INDICATOR DIAGRAMS
TAKEN FROM NO. 2 LAIDLAW ELECTRIC COMPRESSOR,
LONG ISLAND CITY POWER-HOUSE.

FIG. 6.

As there were ultimately two electrically-driven air compressors on the Long Island side, and six low-pressure machines and one combination machine on each side of the river, and as these machines were guaranteed to run at 125 rev. per min. continuously for 24 hours, the maximum free air capacity of all the machines, including the high-pressure compressors, amounted to 102 922 cu. ft. per min. A general view of the Manhattan plant is shown on Fig. 1, Plate XCVIII.

Boiler Plant.—Steam at a boiler pressure of 150 lb. was generated in twelve Stirling water-tube boilers, six on each side of the river, each having a capacity of 500 b.h.p, with 10 sq. ft. of heating surface per horse power, and ¼ sq. ft. of grate surface per horse power. The grates were 8 ft. deep. Each boiler had three steam drums, 42 in. in diameter and 16 ft. 11½ in. long, and one mud drum, 42 in. in diameter and 15 ft. 10 in. long. A manhole was fixed in one end of each drum. The tube plates were ⅝ in. thick.

Each boiler had 405 lap-welded, mild-steel tubes, 3¼ in. in diameter, expanded into reamed holes in the tube plates. The steam outlet was 8 in. and the feed and blow-off valves 2½ in. in diameter. Safety valves, 4½ in. in diameter, were set to blow off at 155 lb. per sq. in.

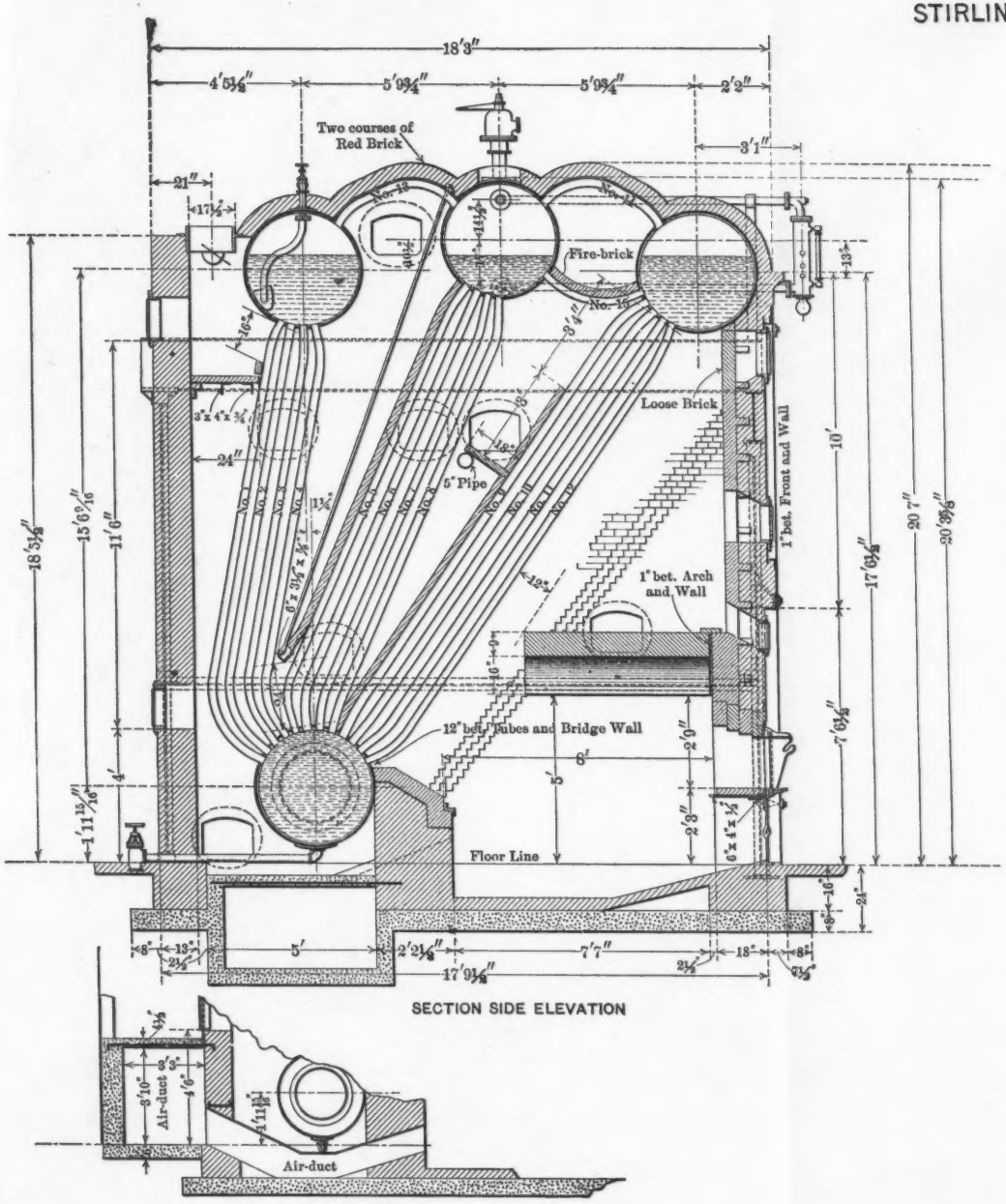
Hydraulic-turbine tube cleaners were supplied with the boilers, and were found to work much more efficiently with high-pressure air than with high-pressure water.

The grates were of the McClave shaking variety. Each boiler occupied a space 19 ft. 9½ in. wide and 18 ft. 3 in. deep, with a height of 20 ft. 11¼ in.

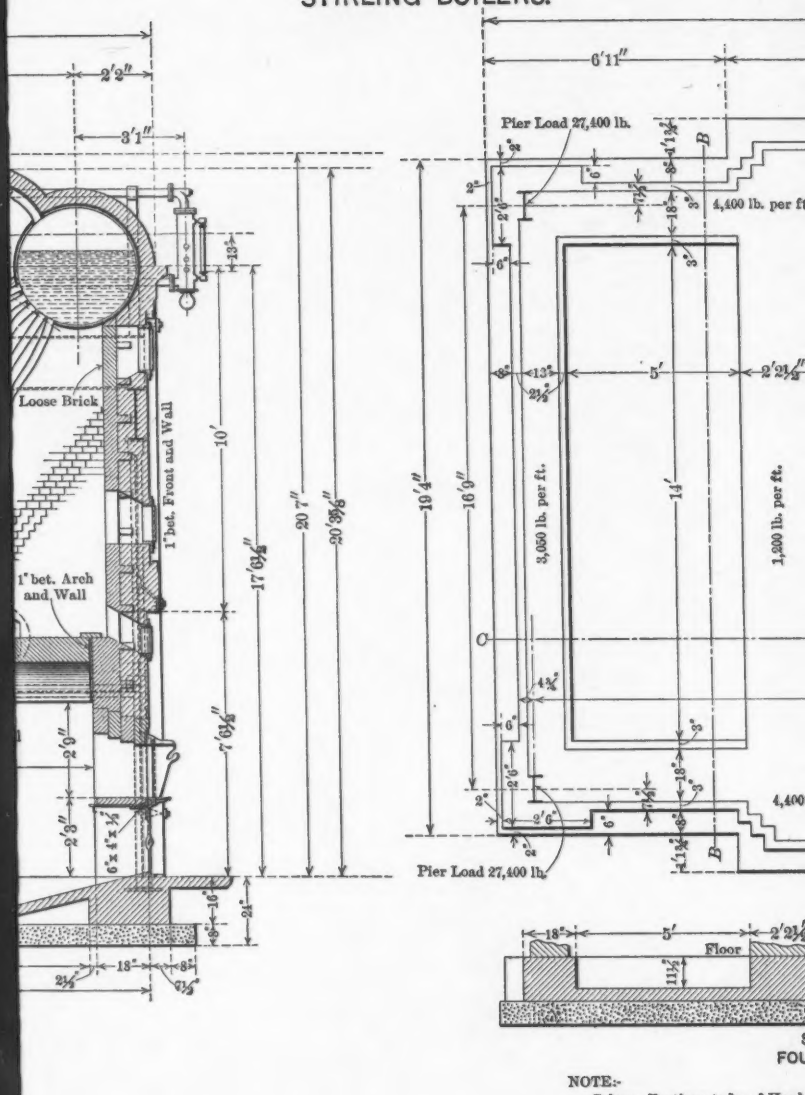
The brickwork was of high-grade fire-brick, 9 in. thick. Before putting the boilers under steam, they were tested to a water pressure of 225 lb. per sq. in.

Each boiler had an independent steel stack, 54 in. in diameter, rising to a height of 100 ft. above the grate level. The thickness of the metal varied from $\frac{5}{16}$ in. for the bottom strakes to ¼-in., $\frac{3}{16}$-in., and No. 8 plates at the top.

The boilers were guaranteed to evaporate 8.7 lb. of water per pound of dry coal having a heat value of not less than 12 000 B. t. u., and not more than 15% ash from and at 212° Fahr., with a pressure in the ash-pit of not less than + 2 in. and a draft at the damper box of 0.75 in. This result was to be obtained with either No. 1 or No. 2 buckwheat anthracite coal. In testing the boilers, they were found


18′3″
4′5½″
5′9¾″
5′9¾″
2′2″
3′1″
Two courses of Red Brick
21″
17½″
No. 13
No. 12
Fire-brick
No. 14
Loose Brick
3″x 4″x ¾″
24″
5″ Pipe
No. 1
No. 2
No. 3
No. 4
No. 5
No. 6
No. 7
No. 8
No. 9
No. 10
No. 11
No. 12
1″bet. Front and Wall
1″bet. Arch and Wall
12″bet. Tubes and Bridge Wall
Floor Line
18′3½″
15′6 9/16″
11′6″
4′
1′11 15/16″
10′
17′6½″
20′7″
20′3⅜″
7′6½″
6′
5′
2′9″
2′8″
6″x 4″x ½″
16″
24″
8″
13″
2½″
5′
2′2½″
7′7″
18″
8½″
17′9½″
SECTION SIDE ELEVATION
Air-duct
Air-duct
Pier Load 27,400 lb.
3,050 lb. per ft.
19′4″
16′9″
13″
6″
2′6″
2″

STIRLING BOILERS.



NOTE:-
Column Footings to be of Hard
carefully laid and bonded.

PLATE XCIX.
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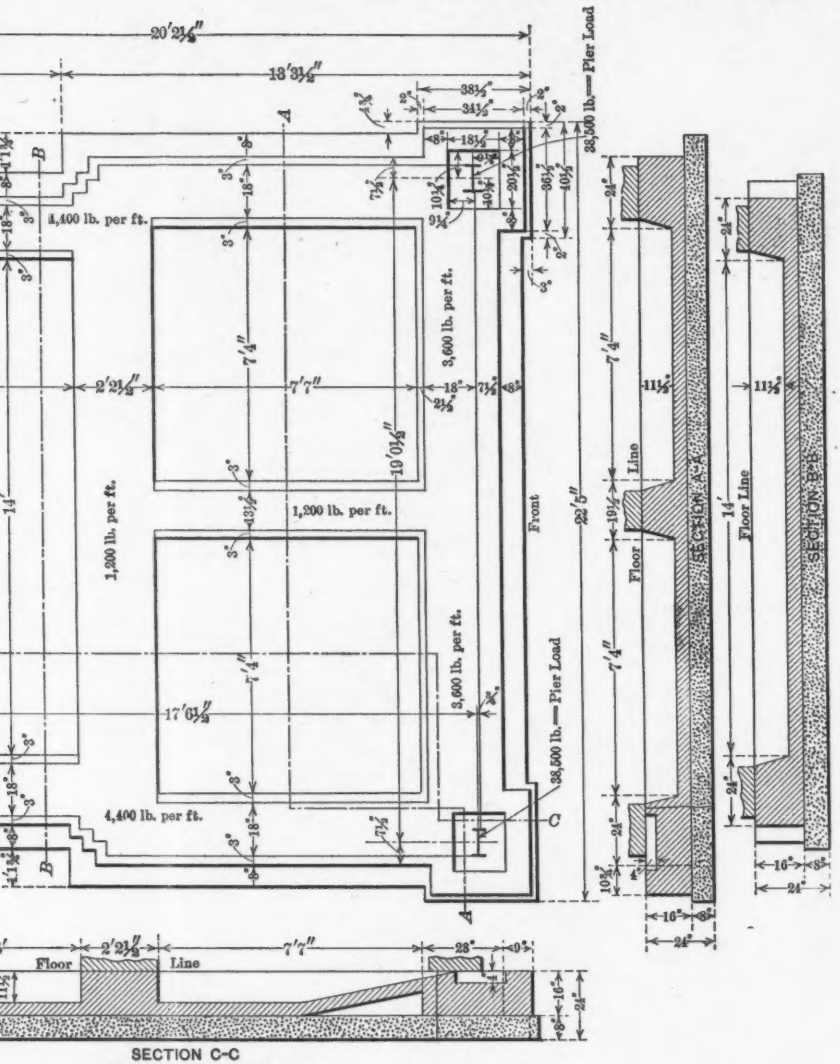

tings to be of Hard Red Brick
ld and bonded.

to exceed this efficiency. The outline drawings, Plate XCIX, show the general arrangement of these boilers.

In computing the boiler capacity necessary, it was estimated originally, before finally deciding on the whole plant, that, on each side of the river, the indicated horse power shown in Table 2 would be developed.

TABLE 2.

	Indicated horse power.	Pounds of steam per indicated horse power-hour.	Pounds of steam per hour.
Electrical plant	580	25	14 500
Air compressors	3 325	15	48 750
Hydraulic, 6 000 lb. pressure	112	25	2 800
Hydraulic, 1 000 lb. pressure	90	25	2 250
Total	4 107	....	68 300

On the basis of 8 lb. of water evaporated per pound of coal, this would represent 8 500 lb. of coal per hour. Assuming 4 lb. of coal per boiler horse power, the capacity would be 2 125 + (25% spare plant = 531) = 2 656; say, five boilers at 500 h.p., or 2 500 b.h.p.

As a check on this, 5 sq. ft. of heating surface will give 1 i.h.p., easily. At sea, with forced draft, it has been done on 2 sq. ft., but say 5, for easy steaming; then the heating surface = 4 107 i.h.p. × 5 = 20 535 sq. ft.; taking 10 sq. ft. per boiler horse power = 2 053 b.h.p., or four units of 500 b.h.p. and one spare, making five of 500 b.h.p. Ultimately, it became necessary to increase the compressor plant, and a sixth boiler was added on each side of the river.

Forced-Draft Apparatus.—Boilers were furnished with closed ash-pits into which entered, for each boiler, two 12 by 36-in. air ducts, from a duct 3 ft. 3 in. by 3 ft. 10 in. Two Buffalo Forge Company's fans were supplied for each plant of five boilers, and another fan was added for the sixth boiler. Each fan was 6 ft. 6 in. in diameter, 30 in. wide at the periphery, 39 in. wide at the center, and had eight blades, driven by a direct-connected, 7 by 8-in., vertical engine above the shaft. The guaranty of the fans is shown in Table 3.

The fans were controlled by regulating valves on the steam pipe; these opened when the steam pressure fell, and allowed the fans to speed up.

In addition to the twelve Stirling boilers, there were on the works

at East Avenue, for driving the headings, four 100-h.p. boilers, of the locomotive type, supplied by the Nagle Boiler Works.

TABLE 3.

Revolutions per minute.	Total pressure, in ounces.	Delivery, in cubic feet.	Indicated horse power.
253	1	27 900	17
284	1¼	31 392	25
313	1½	35 000	33
360	2	39 800	48

The steam per indicated horse-power was not to exceed 40 lb.

These boilers were for supplying steam to the two straight-line compressors already described, and also for driving a fan engine for ventilation, as well as the pumps in the shaft, and the steam derricks. They were of the standard locomotive type, each with a grate 60 in. long and 54 in. wide, and had 88 tubes, 3 in. in diameter and 168 in. long, giving a heating surface of 1 094 sq. ft. Each boiler had an independent smokestack, 30 in. in diameter and 46 ft. high.

These boilers were tested to 150 lb. pressure, and were worked at 90 lb. Other smaller boilers on the different sites were used for steam derricks and heating purposes.

Steam Pipes.—An 8-in. steam pipe led from each boiler into a 12-in. header each end of which joined a 10-in. line led around the power-house to form a loop.

For each compressor a 6-in. branch was taken from the 10-in. header, and, on one side of the 6-in. branch, a 10-in. gate-valve was placed in the header and a 6-in. gate-valve on the branch. Gate-valves were also placed on one side of each 8-in. boiler branch in the main 12-in. header. In this way, if a joint gave out anywhere, it was always possible to repair it after shutting down only one boiler or one compressor.

Independent loops were formed on the steam lines for supplying electric generators, hydraulic pumps, and the auxiliary machinery. Expansion joints were formed by easy loops in the main headers, and Vanstone joints were used in pipes 8 in. in diameter and greater.

The steam pipes were supplied and erected by the Pittsburgh Valve and Foundry Company. This company also fitted the exhaust-steam, compressed-air and water piping, which will be described later, as well as the oiling arrangements.

Each boiler was fitted with a non-return stop-valve, so that, in the event of a boiler tube giving way, the whole battery would not be emptied of steam, but the valve would close when the pressure in the damaged boiler becomes less than that in the main pipe. This type of non-return, crown-head valve was first adopted in battleship construction on account of the danger of a boiler being pierced by a shell, and is now used almost everywhere in boiler plants.

WHEELER'S ADMIRALTY SURFACE CONDENSER

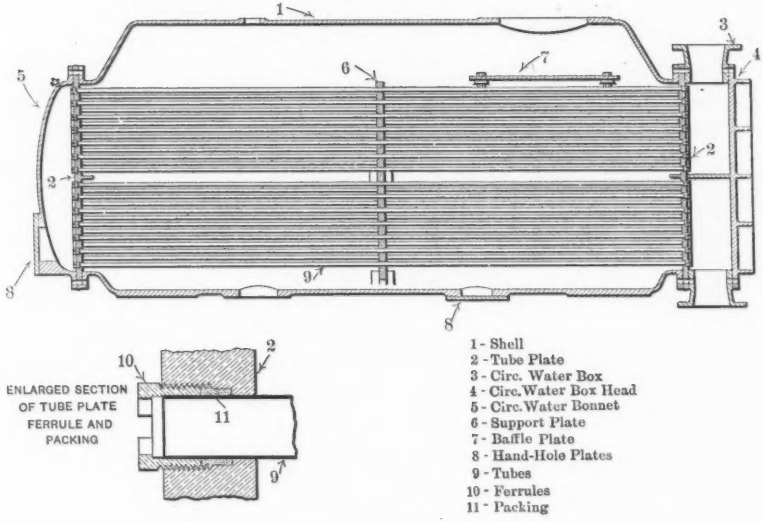

FIG. 7.

Condensers.—Six Wheeler condensers, Fig. 7, were purchased, each with 2 600 sq. ft. of cooling surface. Each condenser had 1 300 $\frac{3}{4}$-in. tubes, of naval composition, namely, 70% of copper, 29% of zinc, and 1% of tin. The body of the condensers was of cast iron. Each condenser was coupled with one Edwards, steam-driven, suction, valveless air-pump, Fig. 8, each having a 7-in. steam cylinder and an 18-in. air cylinder of 12-in. stroke. They were of the double fly-wheel type, single-acting, and dispensing with all suction valves.

Each condenser and pump was guaranteed to be of ample capacity to maintain a vacuum of 27 in. when each unit was caring for 25 000 lb. of steam per hour, with a sufficient quantity of circulating water at 60° Fahr., and 30-in. barometer. Each air-pump was guaranteed to

use not more than 10 h.p., and to consume not more than 60 lb. of steam per i.h.p.

In computing the capacity of the condensers necessary for handling the plant, it was assumed that each square foot of surface would take care of 10 lb. of steam per hour, and, as the evaporation from five boilers amounts to 78 000 lb. of steam, three condensers would each

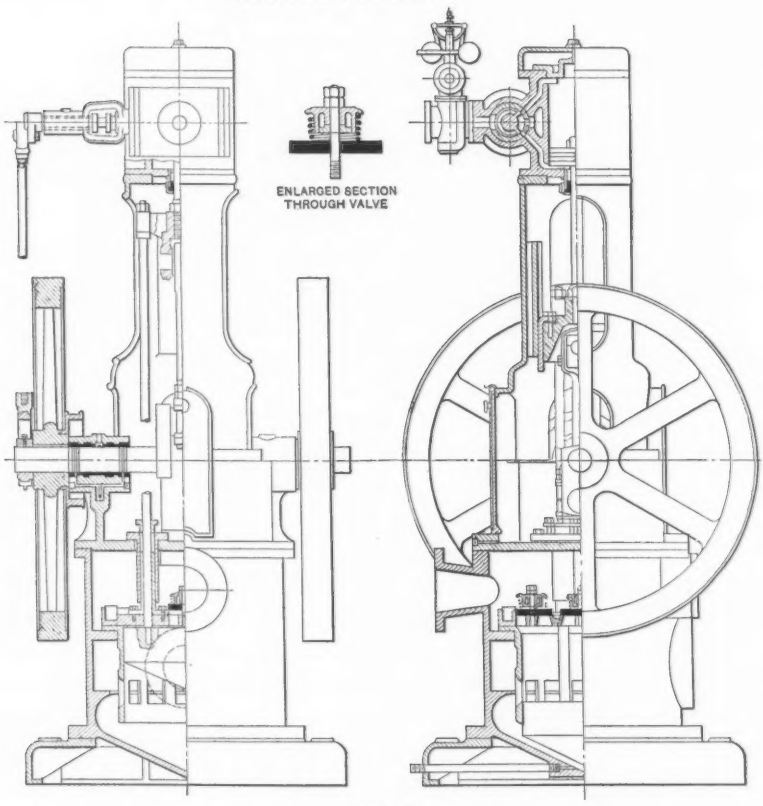

FIG. 8.

have to handle 26 000 lb. of steam, or each required 2 600 sq. ft. of cooling surface. Ultimately, it was decided to install three condensers on each plant of this capacity, and it was found to be ample for the purpose of maintaining a vacuum of 26 in.

In the event, however, of the condensers going out of action or being flooded with steam, and the pressure rising above that of the

atmosphere, two 18-in. atmospheric relief valves were fixed, so that the exhaust steam could pass to the atmosphere.

Each air compressor had a 14-in. exhaust which connected with the main exhaust header. The header was 20 in. in diameter, and had an expansion joint. It connected with each condenser by an easy bend through a 20-in. gate-valve, and reduced from 20 in. to 18 and 14 in. at each end of the power-house, the condensers being located at the middle.

On account of the East River water being used for circulating purposes for the condensers, and from past experiences through the failure of condenser tubes by electrolytic or chemical action, it was decided to protect each condenser tube with a ferrule of zinc, so that the zinc could be attacked in preference to the tubes. The usual naval practice is to suspend sheets of zinc on the water side of the condenser, but ferrules of zinc on split brass tubing, called zincoids, Fig. 9, supplied by the Wheeler Condenser Company, can be placed in each tube where the water enters, and are very effective. So effective were these in neutralizing all corrosion that out of 7 800 tubes only 75 were replaced in four years' operation.

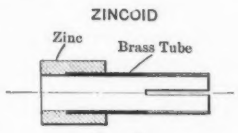

FIG. 9.

Coal Handling Plant.—On the Manhattan side, coal was unloaded from coal scows by a Ransome digger, dumped into 4-yd., steel, side-tip cars, standing on a weigh-bridge, and hauled along the upper staging by a small steam locomotive and dumped in the bunker.

On the Long Island side, it was possible to put in a more economical arrangement, and here the coal was hoisted from the coal scow by a clam-shell and dumped into a hopper from which it was shot into a steel, double-wing, side-dumping car of 24 cu. ft. capacity on a weigh-bridge. The car, on being pushed off the weigh-bridge, was caught underneath on the axle by a hook on a moving chain, and drawn up an incline, from the top of which it ran by gravity into the bunker; the wings of the car were opened automatically at any given point, and the coal dumped.

The car continued to move freely after dumping, and, passing over a pair of points at the end of the bunker, ran up a sharp incline, which brought it to rest and reversed the direction; it then returned down a side road on a down grade to the weigh-bridge.

A bunker with a capacity for 6 weeks' coal supply, on the basis of 600 tons per week, was erected on the Long Island City side, and, on the Manhattan side, where there was not as much room, 4½ weeks' supply was put in. These bunkers consisted of heavy timbers with tie-rods and diagonal bracing. The bottom of the bunkers was on the level of the firing floor, and while the bunkers were well filled the coal slid down on its natural slope to chutes within easy reach of the firemen, who fired by hand. When the supply got short, it was necessary to wheel the coal to the firemen in barrows. There is no doubt that mechanical stokers would have resulted in some economy, both of labor and of coal consumption, but the initial expense was too great for a temporary plant.

On the Manhattan side, the ashes had to be raked from the ash-pits and wheeled to bucket elevators set into the side of the bunker opposite the center boiler, the elevator raising them to a hopper above the upper staging, where they were shot into the 4-yd. coal cars and dumped into scows.

On the Long Island side, where the firing floor was wider, a 30-in. track was laid, and the ashes were loaded into 24-cu. ft., side-dump, steel cars, and hauled up an incline on the dock, where they could be dumped either into scows or cars.

When the boilers were being forced, it was a serious operation to remove the ashes from the ash-pits, and on several occasions, while the tunnels were making their biggest demand for air, they were allowed to accumulate until the grates were burned down. It was a very critical situation, and, after this experience, it would be well in future to mount boilers of this size high enough so that the ashes could be dumped into a hopper below without any danger of burning down the fire-bars.

Coal Consumption.—With five boilers in operation the biggest coal consumption at the Manhattan side for any one month was at the rate of 800 tons per week. This is equivalent to 17 lb. per sq. ft. of grate surface per hour for five boilers. According to the records kept, the average consumption was 2.8 lb. of coal per i.h.p. per hour for all machinery. The coal used on the Long Island side was No. 2 buckwheat; on the Manhattan side, where a greater demand was made on the plant, No. 1 buckwheat was used.

The coal on arrival was tested for its quality by sifting it through

standard screens. No. 1 screen had 2½ meshes per lin. in., or 6¼ meshes per sq. in.; No. 2 screen had 2.84 meshes per lin. in., or 8 meshes per sq. in.; No. 3 screen had 8 meshes per lin. in., or 64 meshes per sq. in. The class of coal supplied did not seem to be according to any fixed standard, nor could the coal merchants be persuaded to specify what was meant by the different grades, named No. 1, No. 2, and No. 3 buckwheat.

No. 1 buckwheat weighed generally 52 lb. per cu. ft. wet, and 49 lb. dry, retaining 70% on No. 1 screen, 15% on No. 2 screen, and 11% on No. 3 screen, with 4% dust. This fell as low as 65% on No. 1, and the dust rose as high as 9 per cent.

No. 2 buckwheat generally retained 8% on No. 1 screen, 23% on No. 2, and 65% on No. 3, with 4% of dust. At times it fell as low as nothing on No. 1 screen, and 10% of dust.

Calorific values were also taken, and generally showed from 11 500 to 12 900 B.t.u., with ash varying from 13 to 20 per cent. The ashes weighed about 46 lb. per cu. ft. wet, and 37 lb. per cu. ft. dry. The actual ash from the boilers was generally from 20% to as high as 30% with a combination of poor coal and inefficient firemen.

Hot Wells.—The condensed water, after being drawn from the condenser by the air-pump, was discharged into the hot well at the general temperature of about 105°, or equivalent to a vacuum of about 27 in. A pump, of the duplex piston type, 7½ by 7 by 12 in., with a stand-by of the same size, was controlled by a float in the hot well to discharge the condensed water into the feed-water heater and oil-separating plant through a 6-in. pipe.

The feed heaters—which were purchased before the oil separators were decided on—were of the Cochrane open type, the exhaust steam from the auxiliaries passing into it and coming in direct contact with the hot-well water. It would have made a better arrangement if a closed feed-water heater had been adopted, and if the feed water, after being purified, had passed through the coils of the closed feed heater with the exhaust steam passing outside them, but, as it was necessary to extract all the oil from the condensed steam, and as the Cochrane heaters were already on the ground, they were installed between the hot well and the oil separator, so that the oil from the auxiliaries might also be removed.

Oil Separators.—The feed-water purifying and oil-separating plant

was a combination of the Masson-Scott purifier, Fig. 10, with the Wilson automatic self-cleansing filter, Fig. 11.

The principle of the apparatus is to treat the oily water from the hot well with alumina-ferric, so that the oil is absolutely separated from the water, and, at the same time, the water is treated with such chemicals as may be necessary for getting rid of scale which might accumulate from the make-up feed. The apparatus consisted of two lead-lined mixing tanks, in one of which the alumina-ferric was dissolved, and in the other soda ash. From this tank the mixture was run into a tank large enough for 24 hours' supply.

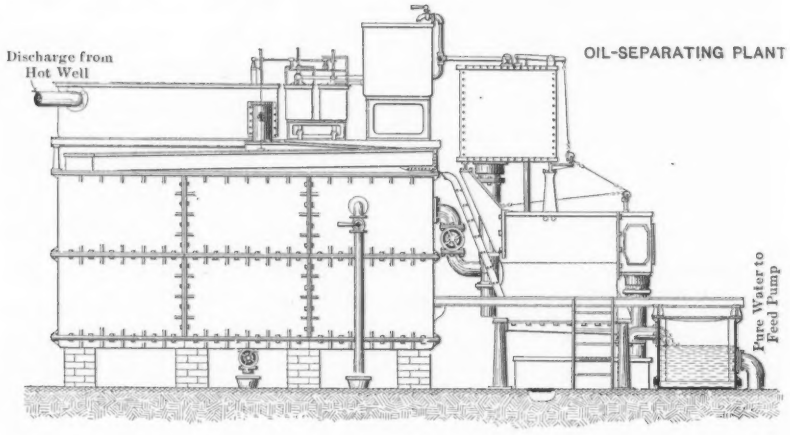

FIG. 10.

The hot-well water discharged into the chemical-regulating tank, which had three compartments. In the first compartment alumina-ferric flowed, in the second a mixture of soda ash, and in the third the hot-well water. A float in the third compartment controlled the outlets of the other two. If there was a large flow of hot-well water, the level rose and the float regulated the quantity of alumina-ferric and soda ash, which flowed with the hot-well water into a mixing trough, fitted with diaphragm plates in order that the ingredients in the water might be thoroughly mixed before entering the reaction tank—a large built-up cast-iron tank, fitted with three diaphragms. The water dipped under the first diaphragm, flowed up over the second, and down under the third, and then left the tank for the filter just under the water level.

The reaction tank was fitted with a large scum-cock, and, as the water flowed slowly under and over the diaphragms, oily matter rose to the surface and was blown off from time to time. The water then flowed through a regulating valve and into the sand filter, which was made in two distinct parts, each capable of taking care of 4 000 Imp. gal. per hour.

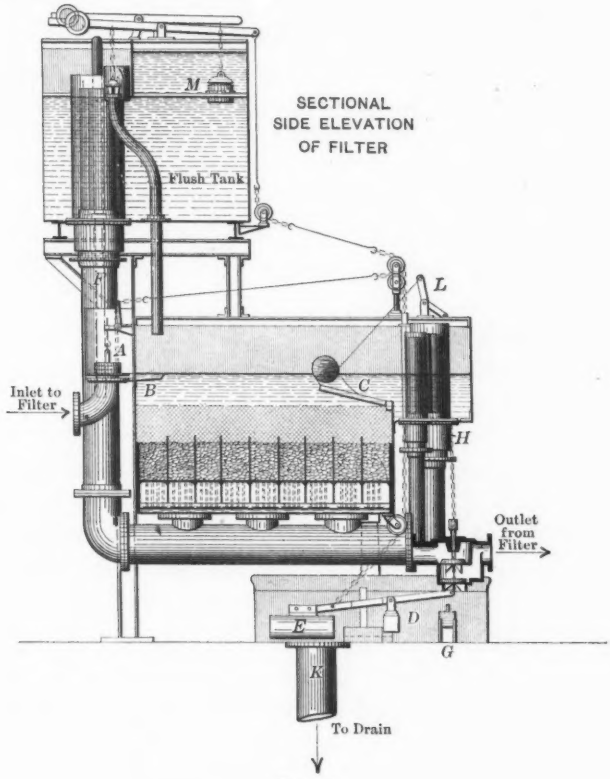

FIG. 11.

The filter, which was built up on cast-iron grids, consisted of a layer of about 9 in. of large pebbles, graduated from large to small, with 7 in. of sand on top. After the water passed through the filter, it was absolutely clear, but in passing through it left behind it a quantity of very fine light particles of oil.

The filtering medium becomes silted up by the impurities arrested from the water, thus preventing the passage of water through the

medium, and causing it to accumulate until it reaches a certain level where it overflows a siphon which draws into a float chamber underneath a sufficient quantity of the water to raise a float. By means of chain connections with the float, the inlet and outlet valves are closed, and a large flushing tank overhead is opened, so that water in great volume is discharged under the bottom of the filter and upward through the filtering medium. The air in the pipes and in the cellular bottom beneath the filter bed is forced through the filtering medium by the down-rush of the water, and aerates and agitates it, and, being followed by the upward rising water, the whole filter is cleansed. The water above the medium rises rapidly, and a much larger siphon goes into operation and carries off the foul water to the sewer; the float chamber, emptying after the rush of water is over, allows the float to fall and resume operations. In order to prevent sand from being carried over into the sewer, a float-board along the front of the filter rises with the water, and, although it is sufficiently below the surface of the water to allow the impurities to pass freely from the filter, the sand which is heavier is prevented from being carried off. The feed water then passes into the feed tank.

Dr. J. W. Schlegel, analytical chemist, of Elmhurst, states that alumina-ferric—more properly called iron alum—is a double sulphate of potassium and iron, viz.: $K_2SO_4\ Fe_2\ (SO_4)\ 3 + 24H_2O$. This acts as a powerful astringent and coagulant, and causes the oil globules to coalesce and rise to the surface. The addition of carbonate of soda, if in large quantities, produces the following reaction: $K_2SO_4\ Fe_2\ (SO_4)_3 + 3Na_2\ CO_3 + 3H_2O = 2Fe\ (OH)_3 + K_2\ SO_4 + 3Na_2\ SO_4 + 3CO_2$, resulting in a voluminous gelatinous precipitate of ferric hydroxide enclosing or forming a nucleus for the oil globules and thus facilitating their separation.

If the chemical strength of the mixture is correct, the proper reaction takes place and the water filters clear. Should the make-up water not be sufficiently hard, that is, should it not have sufficient salts of lime to make a proper reaction, a little soda can be added. As the soda gives an alkaline reaction and the alumina-ferric gives an acid reaction, the quantities of each must be arranged so that the result is neutral in character. This can easily be ascertained by the use of litmus paper.

These oil separators were so effective that no boiler makers were

PLATE C.
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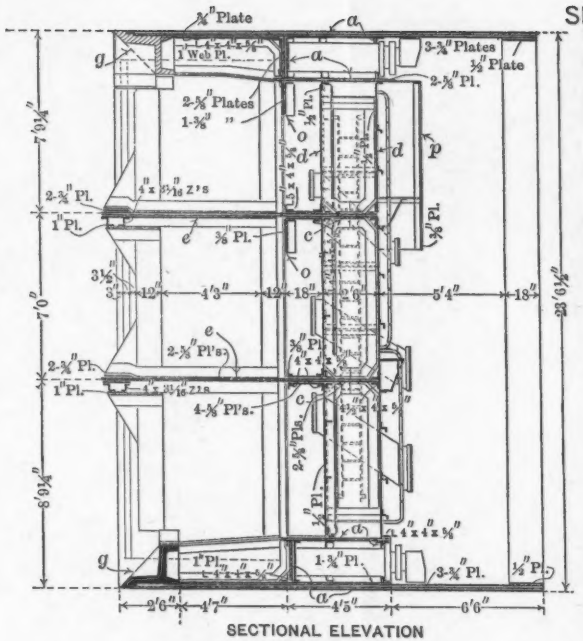
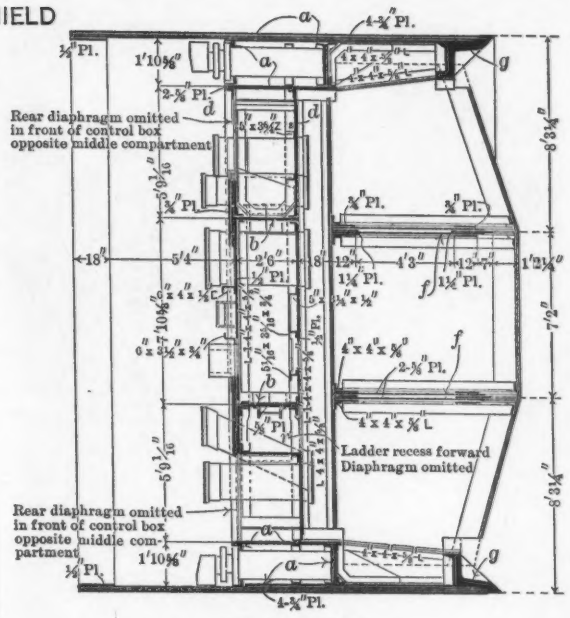
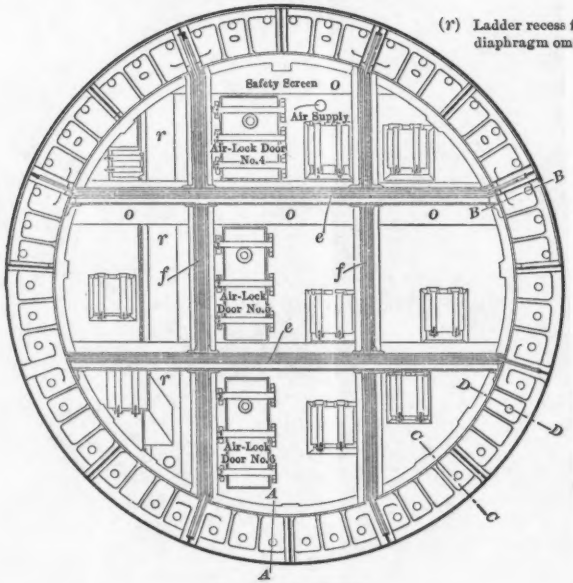
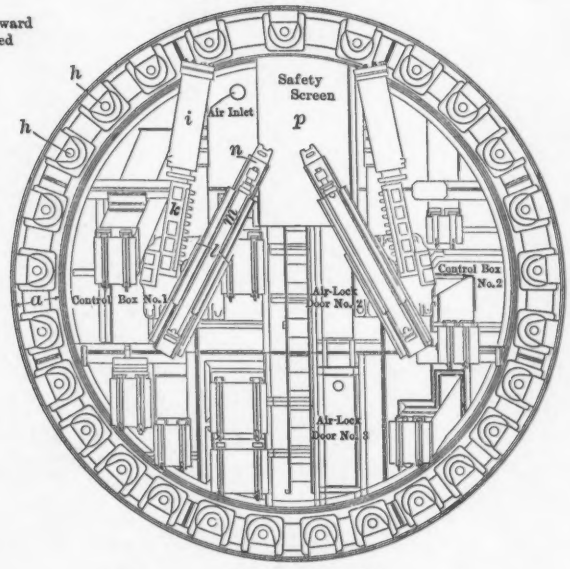
SHIELD





(r) Ladder recess forward diaphragm omitted





employed on the works, and only a few tubes in the twelve boilers had to be replaced.

The water in the feed tank generally stood at a temperature of about 165 or 170 degrees. From the tank it was taken by the boiler feed pumps, which were of the duplex type, 12 by 7 by 12-in. outside-packed, made by the Buffalo Forge Company. There were three of them for each plant, and one pump could take care of the water. There was a regulator on the steam pipe, so that, if all the boiler feed-valves were closed, the excess pressure of the discharge immediately shut off the steam. The boilers were fitted with Reliance gauge columns, which had a float, and, if the water in the boilers rose too high or fell too low, a whistle blew to warn the water tender. Each boiler was also fitted with Murray's automatic feed regulator, consisting of a copper float in a cast-iron box connected at top and bottom to the water columns; if the level fell below normal, a small needle steam-valve opened and admitted steam to the under side of a small piston which opened the feed-valve against the pressure of a spring. When the level rose, the steam-valve closed and the spring shut the feed-valve. These regulators worked very well.

Water tanks were erected with a sufficient capacity for 24 hours' supply, which required 65 000 gal. to take care of make-up feed, hydraulic power, grouting, men's rooms, and sanitation. On the Long Island side, two cedar tanks, 21 ft. in diameter and 15 ft. deep, were erected, and, on the Manhattan side, the same capacity was obtained by putting up seven tanks 11 ft. in diameter, as there was not room for the larger size.

Had tanks been built with sufficient capacity to keep the plant running without purifying the condensed steam and using it for the boiler feed, a capacity of 300 000 gal. additional would have been required.

The general oiling system of the power-house was arranged with separate tanks for cylinder oil and engine oil in the roof of the building, to which the oil was pumped under pressure and flowed through sight-feed lubricators to the different parts of the machinery. The oil that was saved was filtered and pumped back into the tanks. For the air cylinder, lubroline oil was used, to commence with. This oil has a low flash point; viz., 412° Fahr., and, after the engine-room air pressure reached about 38 lb., light clouds of smoke formed in the tunnels

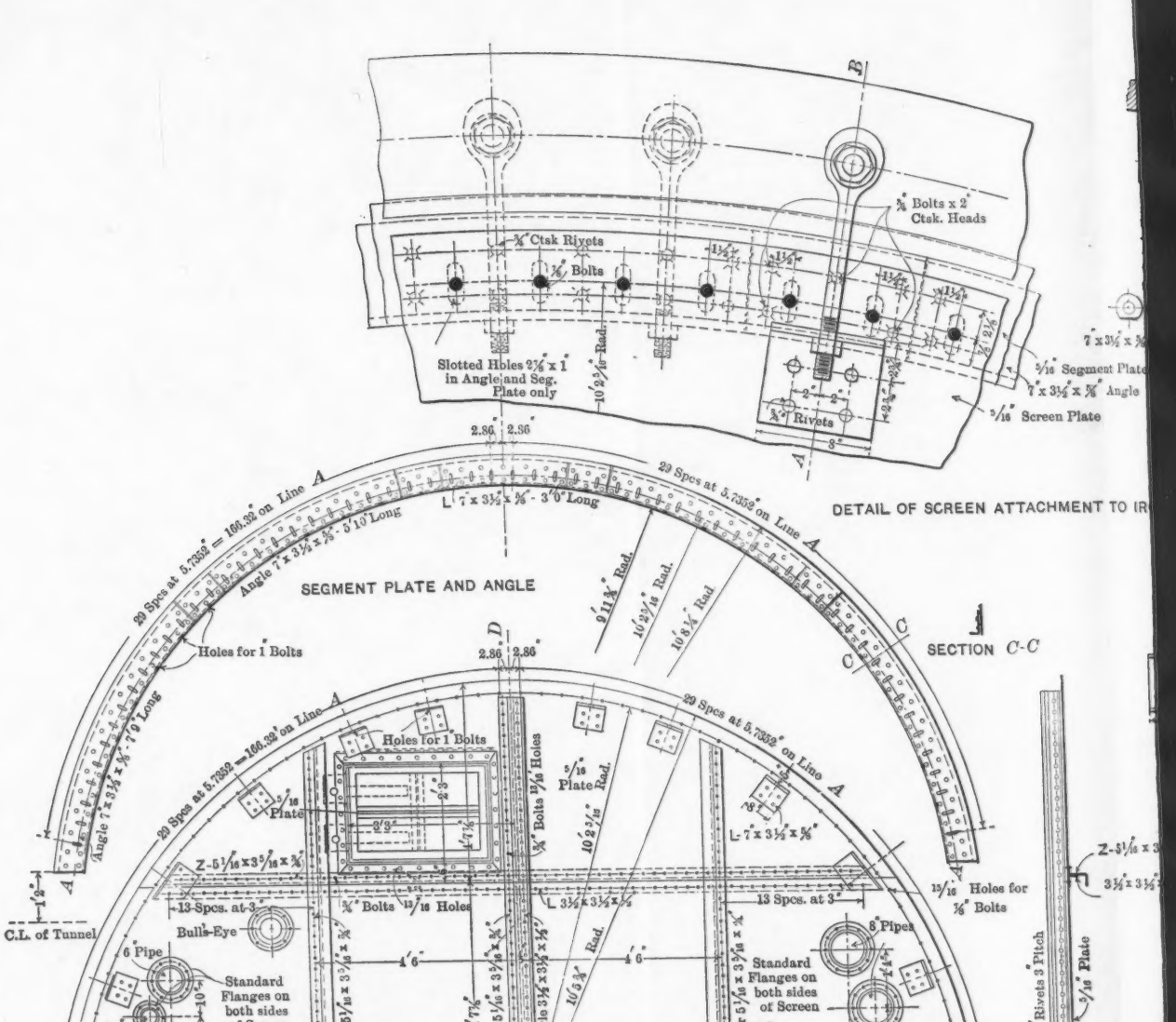
like mist. Charles M. Jacobs, M. Am. Soc. C. E., was consulted about this, and suggested that cheese-cloth be put over the ends of the discharge pipes in the tunnels to observe whether the mist was due to the oil, and in a few moments the cheese-cloth became black.

It was observed that, if this oil was dropped on the outside of the hot-air cylinders, it went off into smoke, and the right-hand high-pressure cylinder of the combination machine at Manhattan exploded, pulling down the air piping with it. This was thought to be due to a spark created by friction in the cylinder igniting the vapor caused by the low-flash oil. Expert advice was obtained, through the Ingersoll Company, which called in Professor Denton, of Stevens Institute, and he advocated the adoption of a heavier oil. An oil named by the Vacuum Oil Company "600 *W* Mineral" oil, with a flash point of 700° Fahr., was then used, and from that time until the end of the work no trouble was experienced; but great care was taken to feed the minimum quantity required, which was small, and a careful examination of the air passages, made since the work was completed, showed only a very slight deposit of carbon from the heavy oil. Examinations were made for this purpose during operation, and the large mushroom-valves showed a deposit of about $\frac{1}{16}$ in. around their edges, but elsewhere the deposit was negligible.

Circulating Water.—The circulating water for the condensers, jackets, and after-coolers of the air compressors was drawn from the East River. On each side of the river was laid one 18-in. cast-iron suction line having a continuous-service foot-valve and a screen which could be raised and cleared without stopping the pumps. This 18-in. pipe was connected by two 10-in. branches with the centrifugals in the power-house used for the condensers. In addition, there was also a 10-in. cast-iron suction line with foot-valve and strainer for the air-cylinder jacket and after-cooler supply centrifugal pump which joined it with an 8-in. suction.

The discharge from the pumps was carried through a 24-in. earthenware pipe to the river.

The condenser centrifugals, of which there were two on each side, were 3 000-gal. Buffalo pumps, able to work against a head of 30 ft., including friction. Each pump was direct-connected to a double-vertical 7 by 5-in. double-acting engine of the enclosed type. They were guaranteed to discharge 3 000 gal. per min. at 350 rev. per min. with


DETAIL OF SCREEN ATTACHMENT TO IR
B
¾" Bolts x 2" Ctsk. Heads
¾" Ctsk Rivets
⅞" Bolts
Slotted Holes 2⅛" x 1" in Angle and Seg. Plate only
10'2⅛" Rad.
1½"
2"
2½"
¾" Rivets
A
3"
7" x 3½" x ⅝"
5/16" Segment Plate
7" x 3½" x ⅝" Angle
5/16" Screen Plate
2.86
2.86
29 Spcs at 5.7359" on Line A
L 7" x 3½" x ⅝" - 3'0" Long
SEGMENT PLATE AND ANGLE
Angle 7" x 3½" x ⅝" - 5'10" Long
29 Spcs at 5.7359" = 166.32" on Line A
Holes for 1" Bolts
9'11¾" Rad.
10'2¾" Rad.
10'8¼" Rad.
C
SECTION C-C
D
2.86
2.86
29 Spcs at 5.7352" = 166.32" on Line A
Holes for 1" Bolts
29 Spcs at 5.7352" on Line A
5/16" Plate
¾" Bolts 13/16" Holes
5/16" Plate Rad.
10'2¾" Rad.
L 7" x 3½" x ⅝"
Z-5 1/16" x 3 5/16" x ⅜"
13/16" Holes for ⅞" Bolts
1'2"
C.L. of Tunnel
13 Spcs. at 3"
13-Spcs. at-3"
Bull's-Eye
¾" Bolts 13/16" Holes
L 3½" x 3½" x ⅜"
8" Pipes
6" Pipe
Standard Flanges on both sides of Screen
4'6"
4'6"
Z-Bar 5 1/16" x 3 5/16" x ¾"
Angle 3½" x 3½" x ⅜"
10'6¾" Rad.
Standard Flanges on both sides of Screen
3½" Pipe
6"
10"
2'10"
5/16" Plate
5/16" Plate
1'6"
1'6"
1'4"
1'4"
¾" Bolts 13/16" Holes
19 Spcs. at 6"
19 Spcs. at 6"
1¼"
1¼"
8'0"
7'7"
2"
2"
2'10"
C.Line of Tunnel
SCREEN
Note: Joints made with Canvas and Red Lead
SECTION D-D
Z-5 1/16" x 3
3½" x 3½" x
¾" Rivets 3" Pitch
5/16" Plate
I-Beam 12" x 3

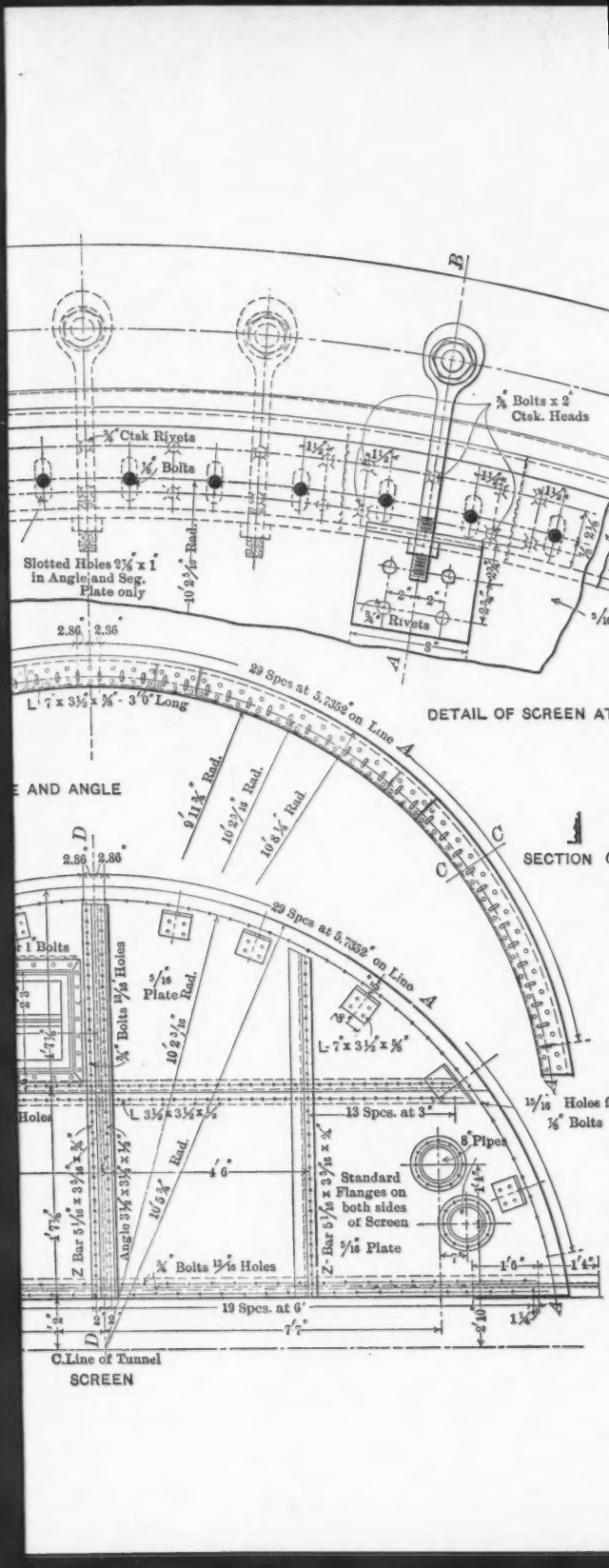
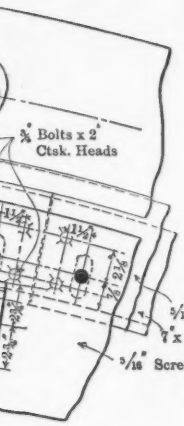
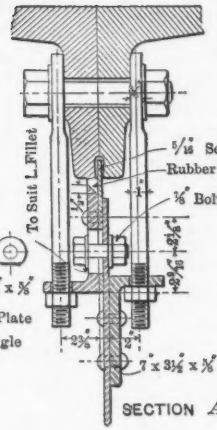

⅞" Bolts x 2" Ctsk. Heads
¾" Ctsk Rivets
⅞" Bolts
Slotted Hbles 2⅛" x 1" in Angle and Seg. Plate only
10'2⅛" Rad.
¾" Rivets
2.86" 2.86"
29 Spcs at 5.7352" on Line A
L- 7" x 3½" x ⅜" - 3'0" Long
DETAIL OF SCREEN AT
AND ANGLE
9'11¾" Rad.
10'2⅛" Rad.
10'6¼" Rad.
SECTION
29 Spcs at 5.7352" on Line A
⅝" Bolts 15/16 Holes
5/16 Plate
10'2⅛" Rad.
L- 7" x 3½" x ⅜"
13 Spcs. at 3"
8" Pipes
Standard Flanges on both sides of Screen
5/16 Plate
Z Bar 5 1/16" x 3 5/16" x ¾"
Angle 3¾" x 3½" x ⅜"
10'3¾" Rad.
4'6"
⅞" Bolts 15/16 Holes
19 Spcs. at 6"
7'7"
C.Line of Tunnel
SCREEN

PLATE CI.
PAPERS, AM. SOC. C. E.
NOVEMBER, 1909.
JAPP ON
CONTRACTORS' PLANT FOR EAST RIVER TUNNELS.

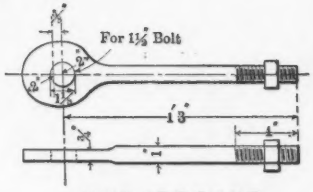
TUNNEL SCREEN



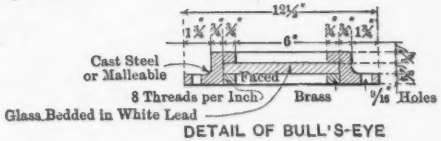
L OF SCREEN ATTACHMENT TO IRON



SECTION *A-B*

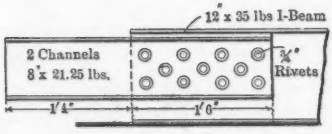


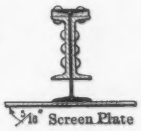
DETAIL OF EYE-BOLT
39 REQUIRED FOR EACH SET



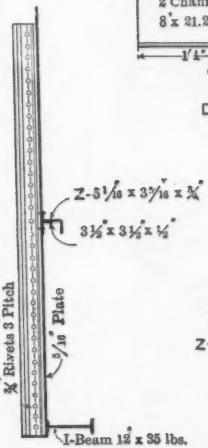
DETAIL OF BULL'S-EYE

SECTION *C-C*

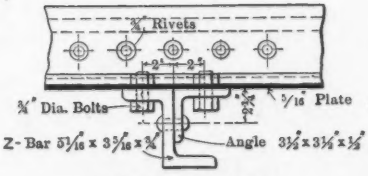




DETAIL AT ENDS OF I-BEAM



SECTION *D-D*



DETAIL OF JOINT

an impeller 30 in. in diameter, and to consume not more than 40 lb. of steam per i.h.p.

These pumps discharged into a 16-in. cast-iron line, having a 12-in. connection to each condenser. For taking care of the jackets and after-coolers, one centrifugal pump having a capacity of 1 000 gal. per min. was fitted on each site. These were driven by 5 by 4-in. double-vertical, double-acting engines, and were guaranteed to work against a head of 40 ft. with an 8-in. suction and discharge. The indicated horse power was 17, giving an efficiency of 60% at 500 rev. per min. The guaranteed steam consumption was 43 lb. per i.h.p.

All impellers were of bronze, and the shafting was of Tobin bronze. A 10-in. cast-iron main was taken from the circulating pump, and branched off to each compressor, with 3½-in. supply pipes to the coolers and 2-in. supply pipes to the low-pressure cylinders. The combination low-pressure was fitted with 1½-in. and the high-pressure with 1¼-in. supply. All discharges from the jackets and coolers were visible, and fell into funnel waste pipes ½ in. larger in diameter than the supply pipes.

The city water supply from the cedar tanks was conducted to the power-house in a 3½-in. pipe, and this was connected as an emergency feed and for make-up feed, and also as emergency coolers and for priming centrifugal pumps. The discharge pipe from the feed pumps was 6 in. in diameter, branching to each check-valve with a 2½-in. pipe. Each feed line was fitted with a 2½-in. safety valve, and with 1½-in. connections for the use of the hydraulic tube cleaners, etc.

High-Pressure Drains.—All drainage points in the high-pressure steam mains and steam separators were connected to the Holly gravity return system, designed by Westinghouse, Church, Kerr, and Company, and shown typically on Fig. 12, which has at its low point a receiver, down into which the drain pipes lead. At an elevation of about 30 ft. above the boiler water line, there was placed a discharge chamber with which the receiver below was connected by a riser pipe.

A direct return line from the discharge chamber restored the water of condensation to the water space in the boilers at substantially the boiler temperature. Pressure in the discharge chamber was sustained by way of the riser pipe, the upward movement of steam raising the water of condensation by entrainment, and with a pressure loss of few pounds.

The discharge chamber was vented for the purpose of clearing the system of accumulating air and rendering the riser action more active, the small amount of vapor escaping being discharged into the feed-water heater.

The whole system operated continuously, without dependence on mechanically-operating elements subject to failure, other than its component piping.

Low-pressure drains were dealt with by steam traps discharging to the hot well.

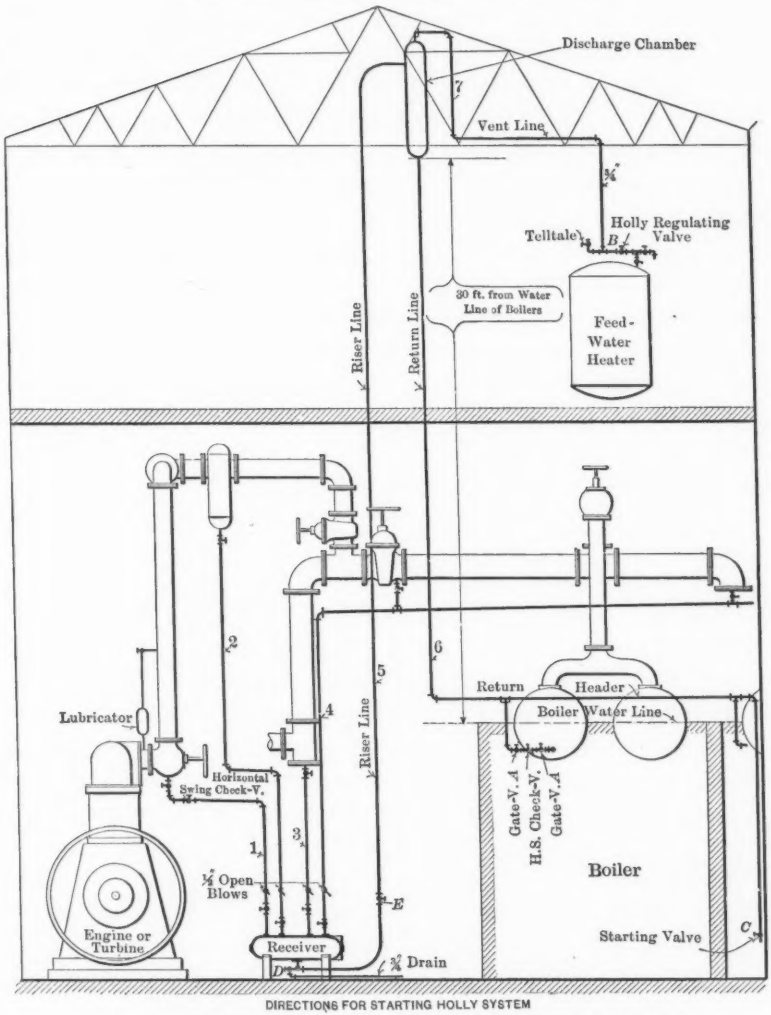
Air Distribution.—The intakes on the Long Island side consisted of an independent 18-in. vertical galvanized pipe for each of the cylinders. Each pipe rose above the power-house roof, and was crowned with a conical top above a wire screen.

On the Manhattan plant, a large galvanized-iron duct was built into the foundations under the compressors, with inlets to each cylinder, and a vertical intake, 2 ft. 6 in. by 1 ft. 6 in., at the west end of the power-house, and a 4 ft. 6 in. by 2-ft. intake at the southeast corner rose above the roof, and turned over with a slight downward bend to prevent rain from entering; the opening was screened with wire netting.

The discharge lines consisted of two easy 12-in. bends from each cylinder to the after-cooler, with a 12-in. connection, between the after-coolers and air receivers, which entered the receiver at the top and left it about one-third of the way up, so that the air flowing down through the height of the receiver deposited some of the moisture and oil which the after-cooler had failed to abstract.

The outlets from the air receivers were 10 in. in diameter, and led to a four-way manifold with a 10-in. sluice-valve on each branch, connecting with a tee to the four 10-in. air lines, one for each tunnel. This made it possible for each compressor to discharge into any or all of the four tunnels, and each tunnel to have any or all of the compressors discharge into it. Four 10-in. air lines, each with two tees and three valves for a by-pass, as specified, led from the power-house into the yard, two going down each shaft. They were cross-connected at the bottom of the shaft, with a sluice-valve between, and one 10-in. main entered each tunnel where it branched off into two 8-in. lines with an 8-in. sluice-valve outside each air-tight bulkhead. Inside the bulkhead, another 8-in. valve was fitted, and beyond it a non-return

HOLLY GRAVITY RETURN SYSTEM
TYPICAL ARRANGEMENT AND STARTING DIRECTIONS



DIRECTIONS FOR STARTING HOLLY SYSTEM

Open return valves, *A-A*, at boilers under steam. Open regulating valve, *B*, slightly. Open valve, *E*, in raiser line. Open all valves in drip lines to receiver. Open starting valve, *C*, until steam blows through, then close. The system is then in operation. When plant is not under steam, open valves, *C*. and *D*

Drip Lines, No. 1, 2, 3, and 4
Riser Line, No. 5
Return Line, No. 6
Vent Line, No. 7

FIG. 12.

valve. Between bulkheads, 8-in. tees with valves were provided from each main for controlling the pressure.

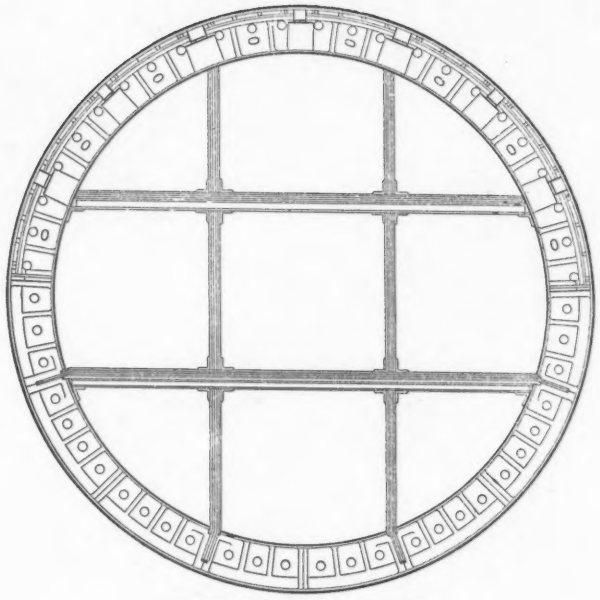
At the delivery end of the 8-in. mains, which were carried up as near the face as possible, light non-return flap-valves were fixed. At the beginning of the work the original specifications were carried out, whereby the air from the mains was connected to the shield by a large flexible rubber pipe, so that the air was blown right into the shield.

It was ultimately found that this was unnecessary, because the same result could be achieved by carrying a blow-out line into the shield, and was much more effectively dealt with than either of these methods by leakage at the face. The 10-in. air mains were standard lap-welded pipe, with extra heavy flanges. The bends and tees were of cast-iron. The 8-in. lines were lap-welded, light-section pipes, with loose flanges and Vanstone joints, made by Stewart and Lloyd, of Glasgow, about $8\frac{3}{16}$ in. inside diameter and $\frac{3}{16}$ in. thick. They were very convenient for handling in the compressed air, as they were very much lighter than the standard lap-welded pipe.

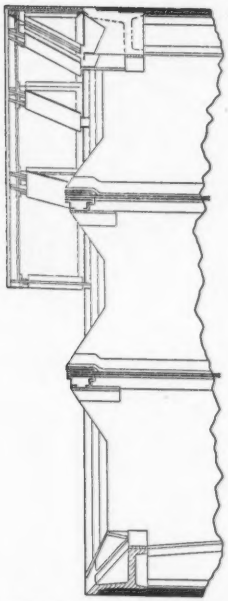
On the Manhattan side, on account of trouble which was experienced through one of the 10-in. lines in the shaft splitting a flange, additional tees were placed at the top of the shafts in the two 10-in. lines, and 8-in. lines were led from these tees to the opposite tunnel to that which the 10-in. line entered, and sluice-valves were placed at the back of each tee. This gave an additional air supply in the event of the vertical pipe being damaged by materials being lowered to the tunnel.

For controlling the pressure in the tunnels, a $\frac{1}{2}$-in. pipe with an ordinary pressure gauge was led from each tunnel to the power-house, and a branch with a recording gauge was led to the office. The gauges in the power-house were fixed on a gauge-board along with recording pressure gauges from each air line, and one mercury gauge on a manifold as a check. The engineer was given the required pressures for each tunnel, and, when it was advisable to regulate the pressure according to the tide, a schedule showing the hourly pressure was used, and a Silvertown electrical tide gauge, controlled by a float in the river, was fixed on the gauge-board in the power-house, and in the office. A supplementary cardboard dial was placed on the tide gauge to show the minimum safe pressure at any tide level; that is, the lowest hydrostatic head to the top of the tunnel.

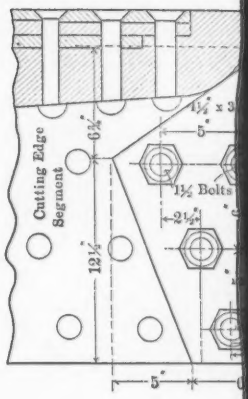
The power-house engineer was able to control the tunnel pressure

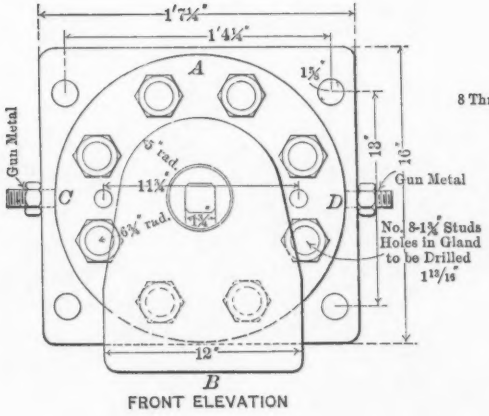


FRONT ELEVATION OF SHIELD
SHOWING FIXED HOOD

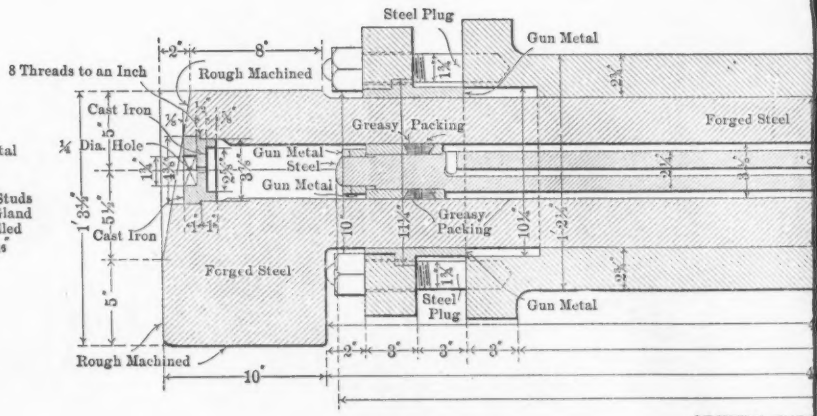


SECTIONAL ELEVATION OF SHIELD
SHOWING HOOD IN POSITION





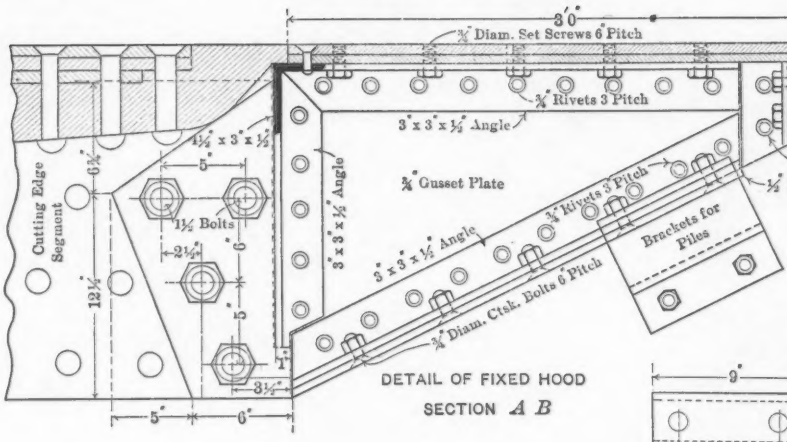
FRONT ELEVATION



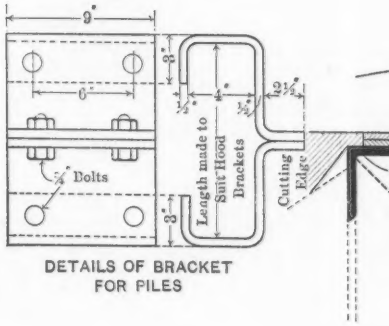
SECTION THRO

FIXED HOOD

VATION OF SHIELD
OD IN POSITION

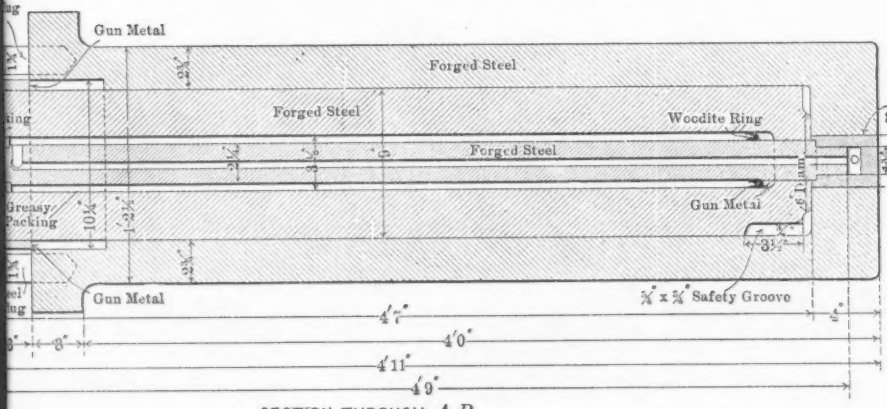


DETAIL OF FIXED HOOD
SECTION *A B*



DETAILS OF BRACKET
FOR PILES

9-INCH JACK



SECTION THROUGH *A B*

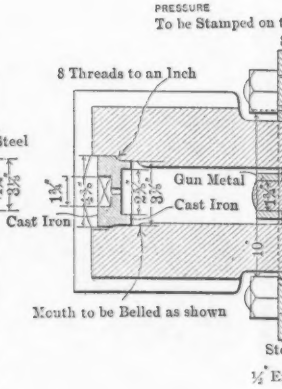
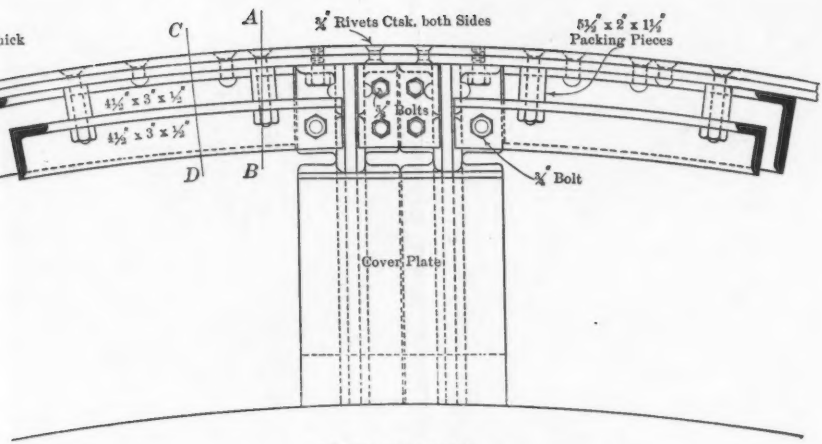
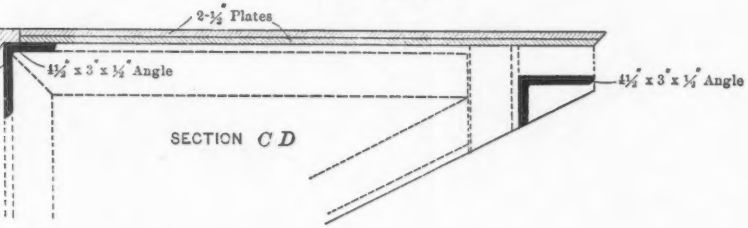


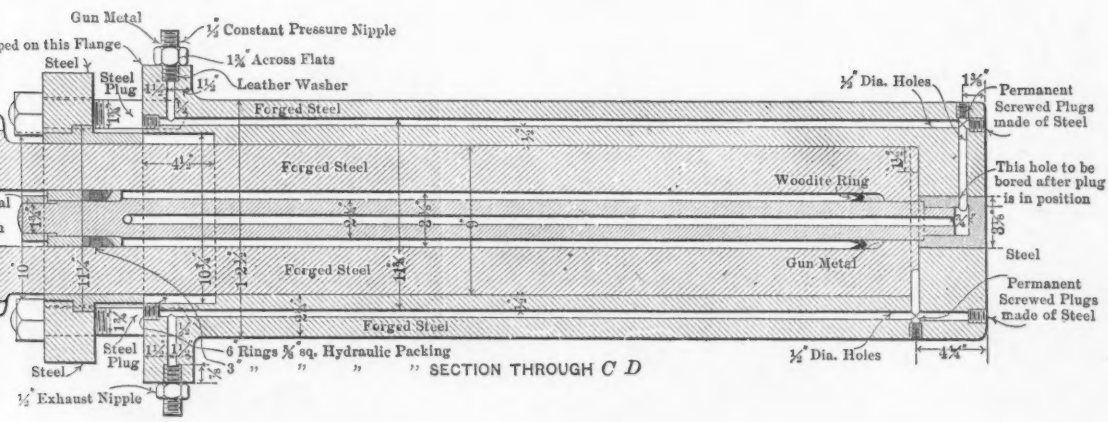
PLATE CII.
PAPERS, AM. SOC. C. E.
NOVEMBER, 1909.
JAPP ON
CONTRACTORS' PLANT FOR EAST RIVER TUNNELS.



FRONT ELEVATION



SECTION *C D*



SECTION THROUGH *C D*

very steadily by the return gauges. Occasionally, confusion would result from the ½-in. pipes being injured and leaking, thus showing a lower pressure in the power-house than was recorded in the tunnel; but, in that event, the telephone system was so complete that the engineer could keep in touch with the walking boss. Telephones to every heading were connected with the main switch-boards in the offices, which had connections to each power-house, and it was possible to communicate with the men in the heading from any point, in connection with the city telephones.

At times as much as 20 000 cu. ft. of free air per minute was forced into one tunnel at a pressure of 35 lb. per sq. in., or an actual volume of 6 000 cu. ft. of compressed air at a velocity of more than 8 500 ft. per min. through two 8-in. pipes. Such a speed involved great friction loss, amounting to as much as from 10 to 15 lb., depending on the length of tunnel constructed. During "blows" requiring such quantities of air, the power-house engineers forced their machinery to the utmost in their endeavor to hold the pressure in the tunnel to the schedule. On these occasions they were surprised to receive orders to reduce the power-house pressure gradually, and more surprised still to find that the tunnel pressure did not fall when the compressors slowed down.

It appeared from this that, after a certain leakage took place, the more air pumped into a blowing tunnel, the more the tunnel would take without improving the pressure. This will appear to be quite logical, within certain limits, after a little thought.

Distribution of High-Pressure Air.—High-pressure air for rock drills, winches, caulking hammers, grout pans, etc., was delivered from the high-pressure air receiver by a 6-in. main which led down each shaft. A 4-in. main led from the 6-in. pipe along each tunnel, and T-connections were put in at suitable intervals along the tunnel, for caulking, grouting, etc. No reheating was attempted. The pipes were standard, lap-welded, with flange connections.

Hydraulic Pressure.—Hydraulic pressure for shoving the shields was supplied at 6 000 lb. per sq. in., and, for the erectors, a pressure of 1 000 lb. per sq. in. was used. The engines adopted for developing this pressure were of the cross-compound, steam, differential, water type, built by the Tannett Walker Company, with 14 and 24-in. steam cylinders, and 18-in. stroke, 2 and $1\frac{3}{2}$-in. differential rams, and 18-in. stroke.

These pumps had a capacity of 6 786 cu. in. at 6 000 lb. per sq. in. developing 60 h.p. at 30 rev. per min., and 18 000 cu. in. at 1 000 lb. pressure, making 80 rev. per min. and developing 18 h.p. Each steam piston had two piston rods on the horizontal center line, which passed on either side of the 2-in. hydraulic plunger and its casing, and were bolted to a heavy cross-head on which were bolted the rams, and a fork connecting rod, bearing on extended cross-head gudgeon pins, passed over the casing of the $1\frac{13}{32}$-in. ram to the cranks on either side of the fly-wheel. The plungers were outside-packed, and, owing to the differential action, they required the same effort on each stroke and only required one suction valve and one discharge valve for each pair of differential rams, the 2-in. ram being just twice the area of the $1\frac{13}{32}$-in. ram.

The number of engines and the capacity were computed on the assumption that the full $2\frac{1}{2}$-ft. stroke of 27 jacks, 9 in. in diameter, on four shields, with 10% leakage, would be required in $\frac{1}{2}$ hour, and amounted to 7 600 cu. in. of water per minute.

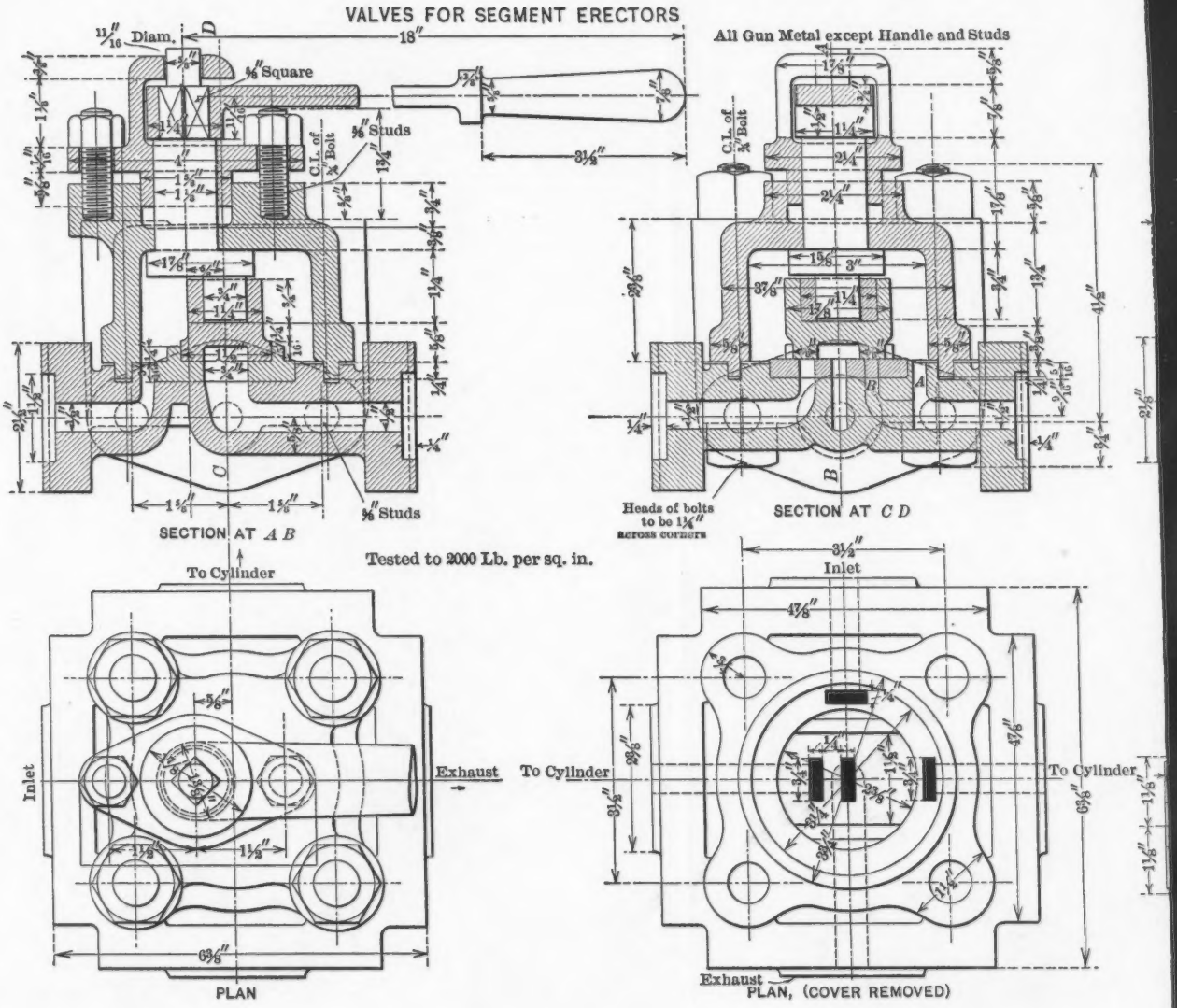
For low-pressure water, the complete movements of the erectors for placing a ring of iron were calculated. The erector, having an $11\frac{1}{2}$-in. piston on an $8\frac{1}{8}$-in. ram with a $3\frac{1}{2}$-ft. stroke, and the push and pull for placing the plates having a $4\frac{1}{4}$-in. piston on a 3-in. ram with a stroke of 6 ft. 2 in., the complete movements for two erectors on one shield, with 10% added for leakage, assuming that the whole ring could be placed in $\frac{1}{2}$ hour, amounted to 2 020 cu. in. of water, and, with the four shields, 8 064 cu. in.

It was decided, therefore, to purchase three Tannett Walker engines, as described above, two of them being used for high pressure, giving a capacity of 13 572 cu. in. displacement as against a requirement of 7 600, and one being used for low pressure with a capacity of 18 000 cu. in. as against 8 084 required, so that practically double the calculated capacity was installed.

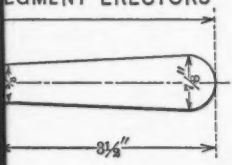
The high-pressure hydraulic system consisted of a $1\frac{1}{8}$-in. inside diameter and $1\frac{15}{16}$-in. outside diameter, extra heavy, American, hydraulic pipe, built up of three different thicknesses of pipe shrunk one on the other, joined by heavy cast-iron couplings. It was tested in the yard to 6 500 lb. per sq. in. before being sent to the tunnel.

From each of the three engines on each side of the river a $1\frac{1}{8}$-in. pipe, having a screw-down stop-valve, was joined to a manifold with

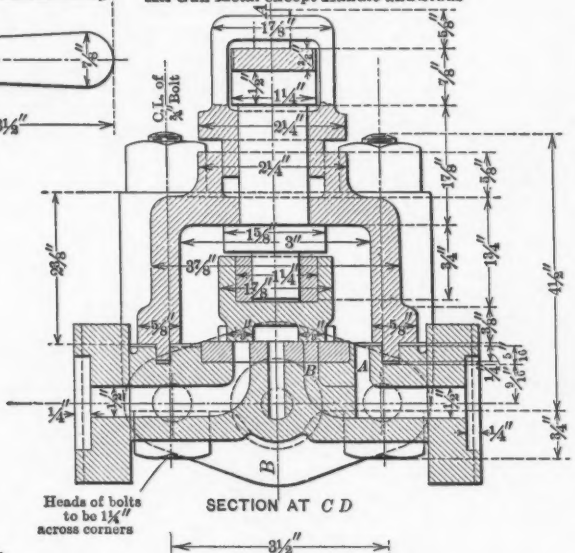
VALVES FOR SEGMENT ERECTORS



EGMENT ERECTORS

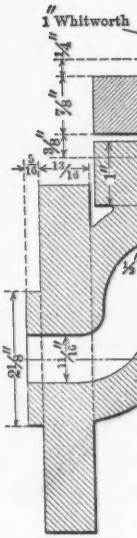


All Gun Metal except Handle and Studs

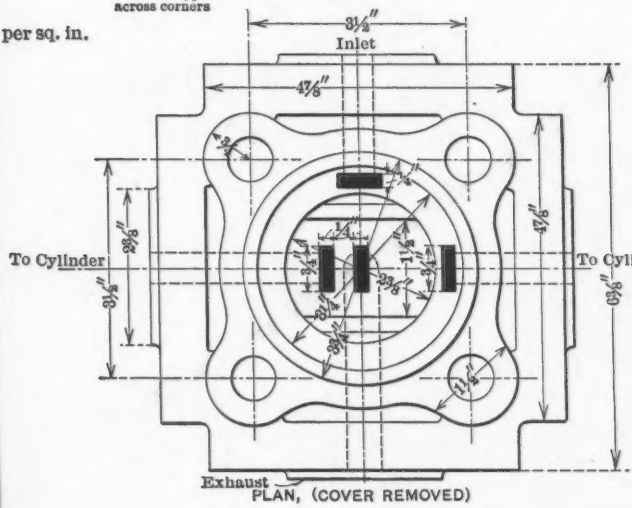


Heads of bolts to be 1¼" across corners

SECTION AT *C D*



. per sq. in.

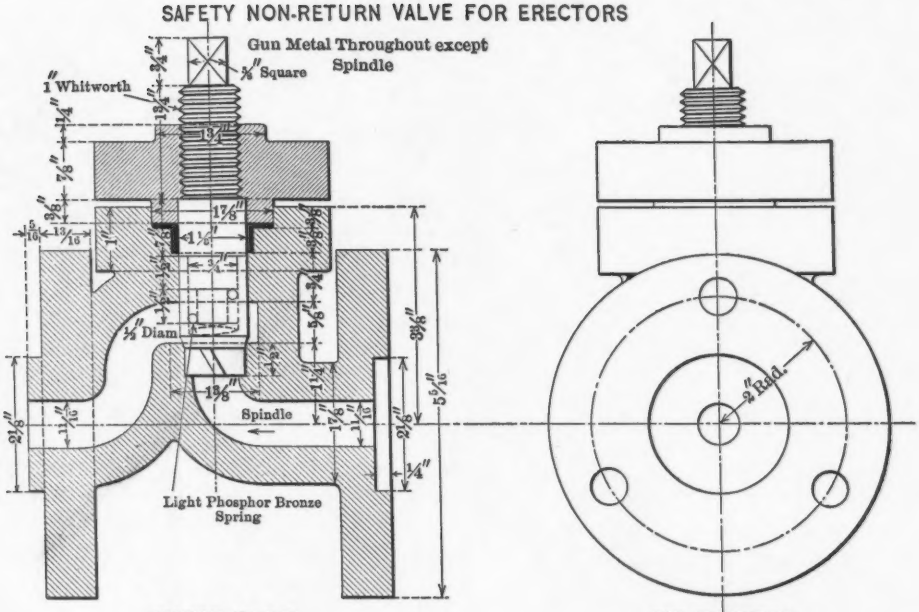


Exhaust

PLAN, (COVER REMOVED)

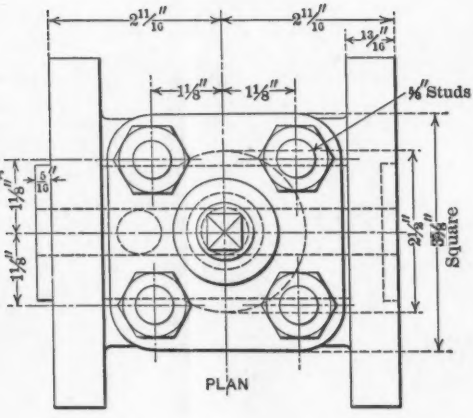
PLATE CIII.
PAPERS, AM. SOC. C. E.
NOVEMBER, 1909.
JAPP ON
CONTRACTORS' PLANT FOR EAST RIVER TUNNELS.

SAFETY NON-RETURN VALVE FOR ERECTORS

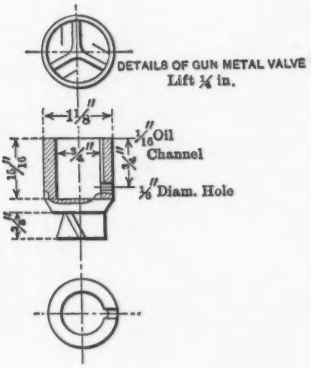


SECTION AT EF

END ELEVATION



PLAN



four branches. Each branch had a screw-down valve, and led to one of four main high-pressure lines leading to the shafts; two of the pipes led down each shaft and were joined at the bottom, forming a loop.

From each end of the loop a 1⅛-in. line entered each of the two tunnels at the foot of the shaft, and on either side of the connections a screw-down valve was fixed in the loop as well as on the line entering the tunnel. By this means it was possible, if one of the lines to the shaft burst, to use the other.

The pressure line along the tunnel was connected to the valve box in the shield by a flexible high-pressure armored-hose pipe. These frequently gave out, and, as often as not, the connection was made with a piece of copper pipe of sufficient length to allow the shield to move forward without breaking the joint. On each main was fitted a safety valve, so that if the pressure at the shield was shut off suddenly the stored energy of the fly-wheel would not burst the pipes. A by-pass near the shield was opened before the pressure was shut off the jacks.

The low-pressure hydraulic system consisted of two lines of pipe 1⅛ in. in diameter inside and 1¾ in. outside, with male and female joints, and leather gaskets held by 5½-in. flanges with three bolts ⅝ in. in diameter.

All three engines were coupled, through a stop-valve, to a loop formed by joining the two low-pressure mains, and a valve at each end of the loop cut off either of the two lines.

From each line connections controlled by stop-valves led to the accumulator and down each shaft, where a loop was formed, as on the high-pressure line, from which the low-pressure mains led into each tunnel. The accumulator supplied the erectors with a constant pressure, so that, if the engine speeded up or stopped suddenly, the erector would not lash about and injure the workmen.

As the total quantity of water required for the building of one ring in four tunnels simultaneously was 60 634 cu. in., including 10% loss, it was decided that the accumulator ought to supply one-third of this capacity. This gives the engine considerable latitude. The accumulator, therefore, was made with a capacity of 20 358 cu. in., or 12 in. in diameter and 15 ft. stroke. One was ordered for each side of the river from R. D. Wood and Company, of Philadelphia.

The accumulators were designed with the plunger fixed to the base

and standing upright. The cylinder which surrounds the plunger was tested to a water pressure of 3 000 lb. per sq. in. before erection. Around it was built a steel tank of sufficient size to load the accumulator to 1 000 lb. per sq. in. when filled with material weighing not less than 100 lb. per cu. ft.

Castings bolted to the side of this circular ballast tank slid on vertical timber guides, and the whole plant was housed in a steam-heated building. It was connected to the steam stop-valve of the low-pressure hydraulic engine by a light wire rope, so that, when it fell, the stop-valve opened and then closed when the accumulator rose near the top of the stroke, and, if it continued rising, a rigid steel rod attached to a relief valve was lifted to prevent over-stroking.

It was found necessary for a time at Manhattan to supplement the three pumps with an electrically-driven Watson Stillman pressure pump. This was made strong enough for 6 000 lb. per sq. in., but was used almost exclusively at a pressure of 1 000 lb., coupled with the accumulator. It was fitted with four rams, $1\frac{1}{4}$ in. in diameter and having a stroke of 6 in., and driven through gearing by a 50-h.p. Westinghouse motor, built for 250 volts.

On the Long Island side, for the same reason, it became necessary to supplement the low pressure by means of one belt-driven gear, three-throw, 6-gal. per min., Berry pump, with rams 2 in. in diameter and 6 in. stroke, driven by a 30-h.p. motor.

For a time, this pump and another of the same type, but two-throw, were used at East Avenue for one of the shields, and later a Watson Stillman air-driven, duplex, 5 500 lb. pump, with two air cylinders, 14 in. in diameter, and differential duplex water cylinders $1\frac{1}{2}$ and $2\frac{1}{8}$ in. in diameter and 12 in. stroke, was used for one of the East Avenue shields. It was found convenient to mount this on the traveling staging at the back of the shield. This saved long lengths of hydraulic pressure pipes through the tunnels.

Fire Service.—Fire service was provided at each site, with a 4-in. main in the yard and $2\frac{1}{2}$-in. stand-pipes; 50-ft. lengths of hose pipes, with standard connections and nozzles, were fixed on each pipe and folded up in a box. Water was supplied to this system by special meter, and connection was also made to a high-pressure service pump in the power-house, with a salt-water suction. The main also entered the men's rooms and offices. In the tunnels, the same fire service was

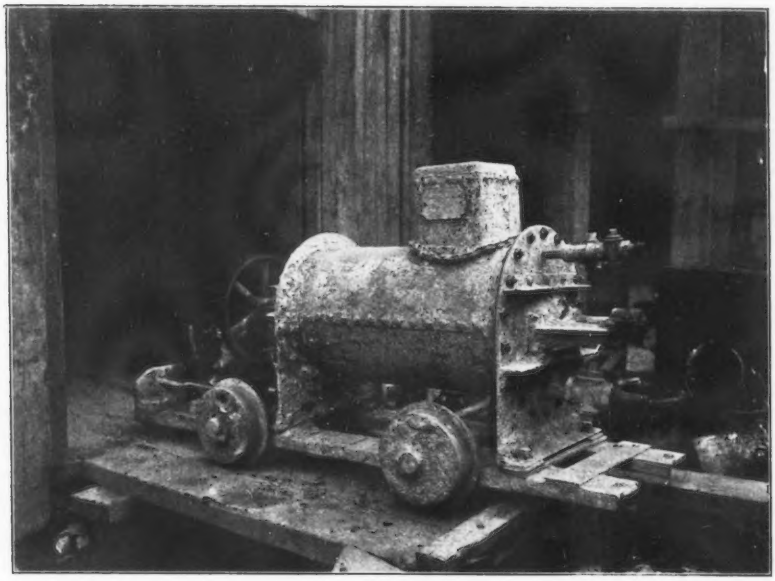


FIG. 1.—GROUT PAN.

FIG. 2.—HAULAGE GEAR.

available, with connections every 200 ft., and at the shield and bulkheads, where there was the most likelihood of fire, there was, in addition, a connection on both hydraulic services. In each air-lock a fire connection was fixed, with a short length of hose ready for use, and, in addition, a high-pressure air connection was led to each air-lock. The air connection was adopted after a serious fire had occurred in one of the tunnels on an off Sunday through the carelessness of the tunnel watchman who was killed by fire and smoke, evidently while asleep on duty. The men attempting to get to work on Monday morning were suffocated by the hot and smoke-charged air entering the lock from the tunnel. A high-pressure hose pipe was connected to the decompression cock and the air-lock was filled with fresh high-pressure air, so that the men were able to get inside the tunnel to put out the fire.

The high-pressure air combined with the fire-hose made it possible to control a fire near the bulkhead. In addition to these precautions, each bulkhead was fitted with a 6-in. pipe mounted with three double-standard Siamese fire connections, with non-return valves, on the outside or atmospheric side of the bulkhead, and, on the pressure side of the bulkhead, there were four standard fire connections, so that in the event of an uncontrollable fire, the fire brigade could couple high-pressure hose pipes and flood the tunnel.

After one of the powder magazines, with about 800 lb. of 60% dynamite, had burned up, without any known reason, while the powder watchman was beside it, it was decided to fix in the roof of each powder magazine a 4-in. connection with a valve controlling it at a safe distance away, so that if a fire started in the magazine or anywhere near it, the valve could be opened and the powder magazine kept practically full of water.

In connection with the powder magazines, which come under the Fire Department, the detonators were always kept some distance from the magazines, which were of the standard approved type adopted in New York City.

For the fire protection of oil stores, the roof of each oil store had a very thin matchboard lining, about 8 in. below the roof, and the space between was filled with sand, so that if the oil caught fire, the ceiling would very quickly burn through and dump the dry sand on top of the oil.

On the Manhattan side, where repeated attempts were made by

incendiaries to burn down the office buildings and the men's rooms, two 3-in. perforated pipe lines were run above the ceiling of each building, as a precaution, so that in the event of a fire the building could be drenched. The usual chemical fire extinguishers prescribed by the underwriters were placed in all buildings.

Electrical Apparatus.—In determining the amount of electrical energy necessary, calculation was made of the horse power required for the various machines, which were to be driven by electricity.

The original estimate for each site covered:

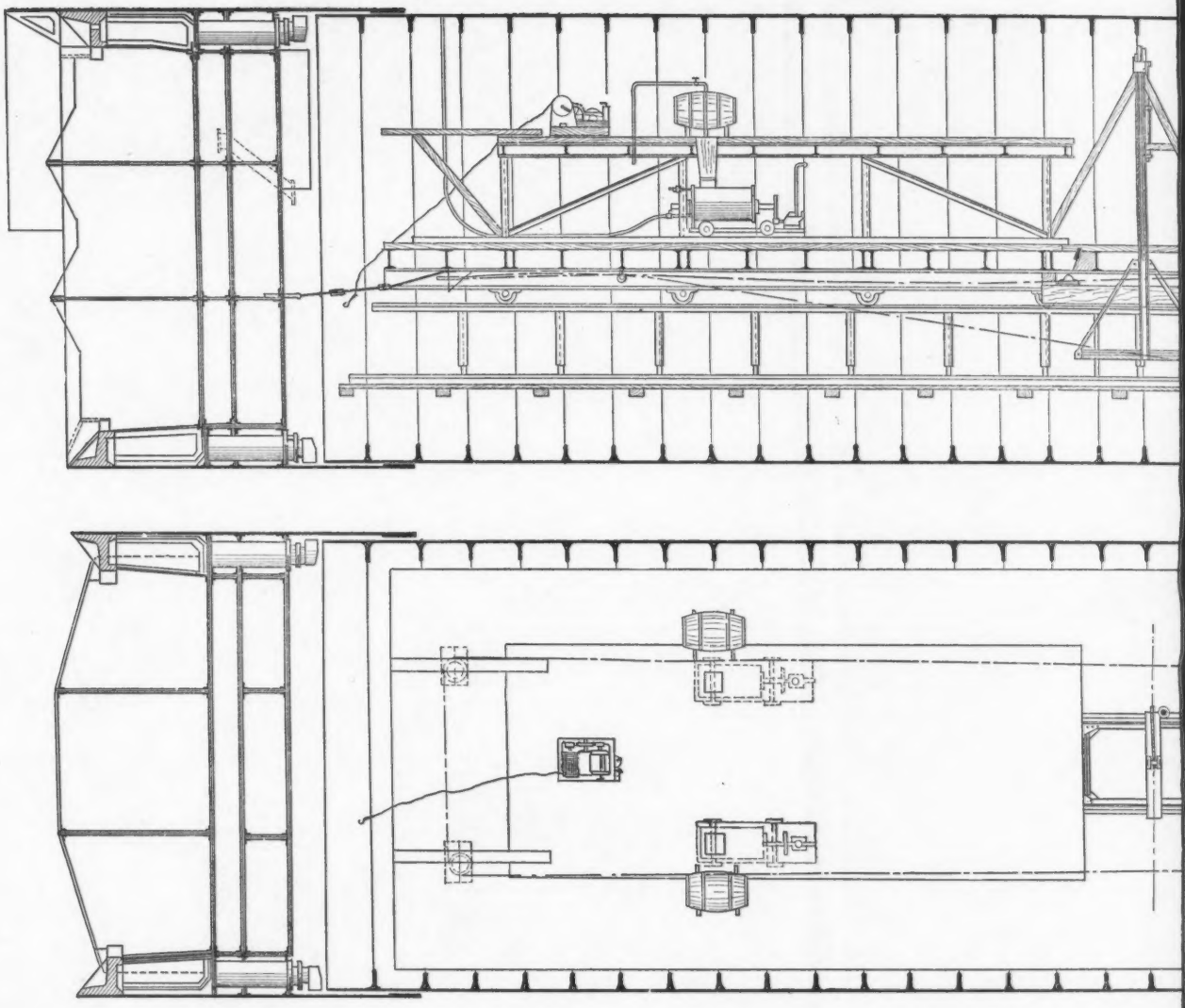
Two hoists at 30 h.p.	60 h.p.
Four haulage gears at 30 h.p.	120 "
Electric light for offices and tunnels	85 "
Stone crusher	40 "
Concrete mixer	30 "
Pug mill	15 "
Work shop	30 "
Total	380 h.p.

Estimating the mechanical efficiency of the engine and dynamo at 90%, and the efficiency of the motors at 90%, gave a combined efficiency of 81%, or a total horse power of 470 for each working site; but, as all this plant could not be in operation at the same moment, it was decided to consider that not more than 50% would be demanded at one time, making the requirements 235 h.p. for each site, or 179 kw.

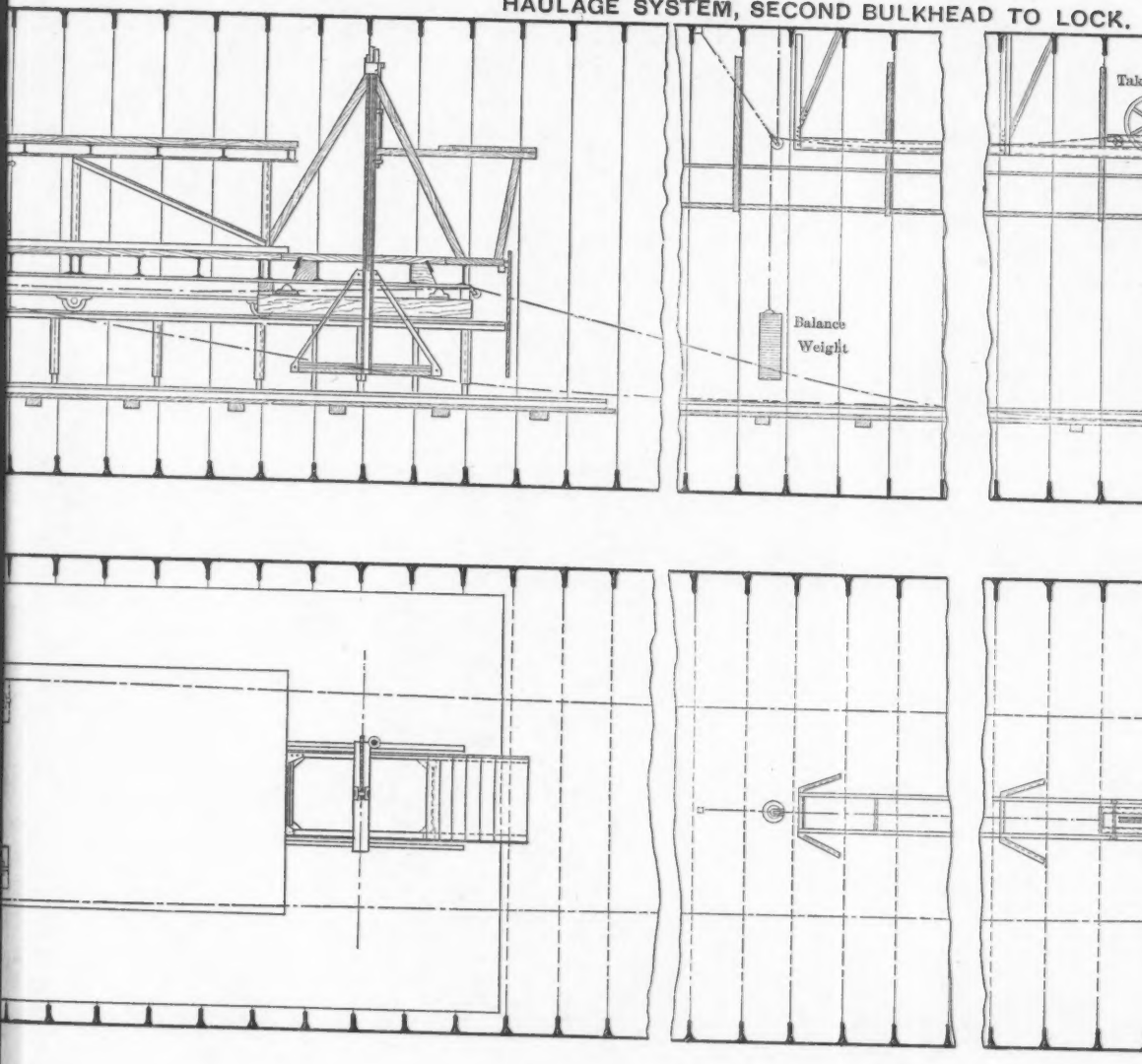
For the Manhattan side, three units of 75 kw. each, and for the Long Island side, which also had to take care of the East Avenue site, three units of 160 kw. were purchased.

On account of the limited ground space available, vertical engines were decided on, and those purchased were the Reeves cross-compound vertical engines, direct-connected to General Electric generators. The larger units had engines of 12½ and 20 in. in diameter with 16-in. stroke, developing 225 i.h.p. at 230 rev. per min., and the Standard General Electric dynamo of 160 kw., 640 amperes, 250 volts, at 230 rev. per min.; the smaller units were 8½ and 17 in. in diameter with 12-in. stroke, at 270 rev. per min.

The speed of the engines was guaranteed to vary not more than 1½%



HAULAGE SYSTEM, SECOND BULKHEAD TO LOCK.



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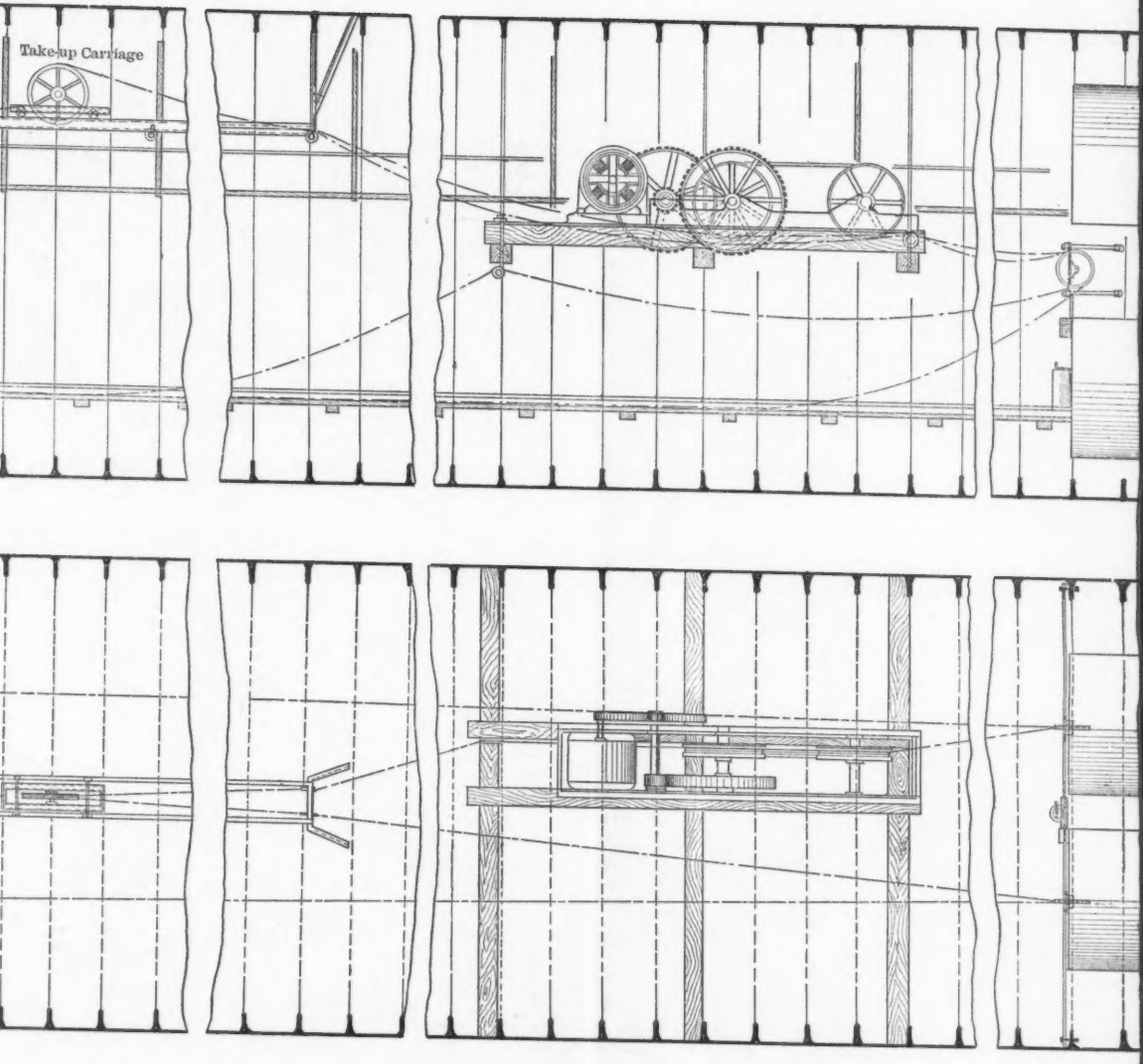
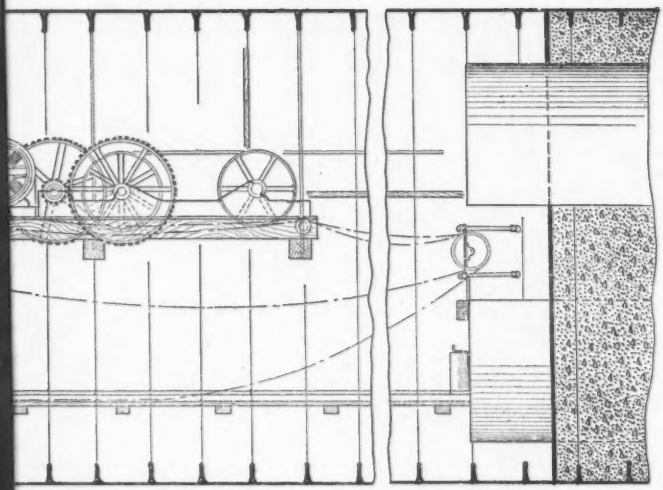
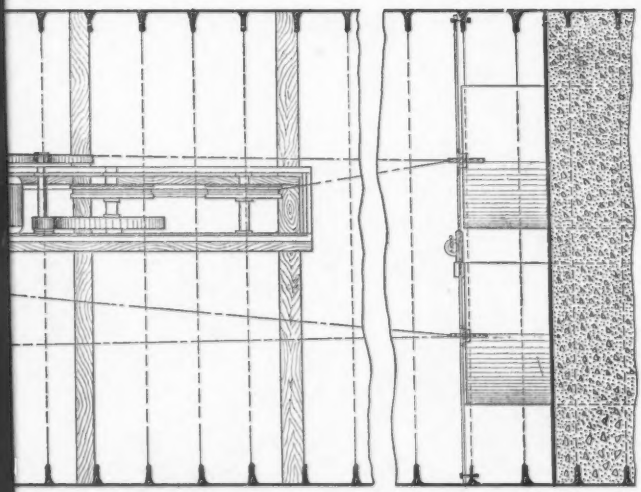
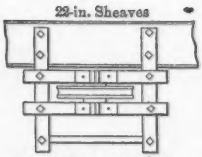
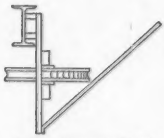




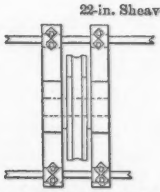




FRONT ELEVATOIN.
UNDER GROUT PLATFORM.



SIDE ELEVATION



FRONT ELEVATOIN.
AT BULKHEAD

SIDE ELEVA
AT BULKHE

PLATE CV.
PAPERS, AM. SOC. C. E.
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JAPP ON
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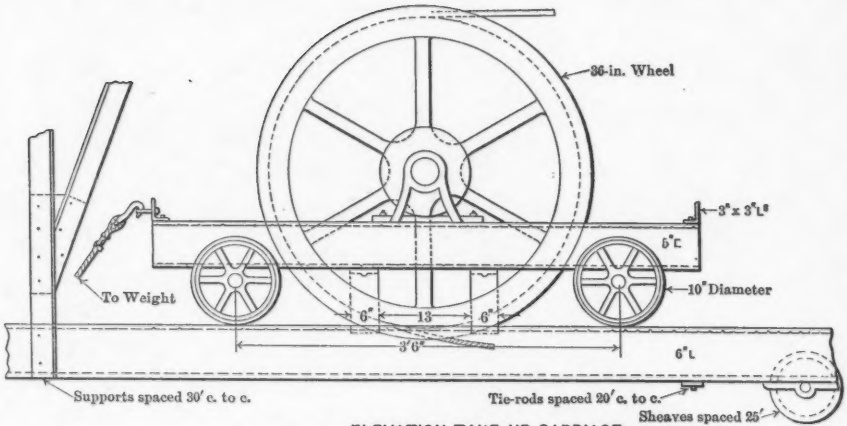
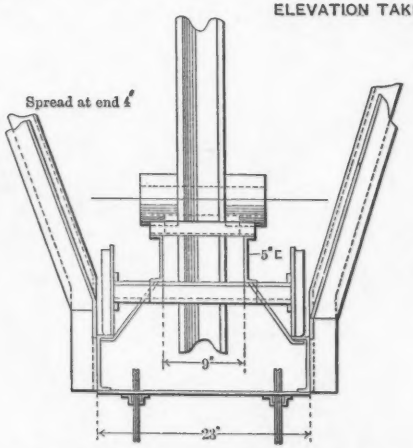


ELEVATION TAKE UP-CARRIAGE

SIDE ELEVATION
AT BULKHEAD



FRONT ELEVATION OF TAKE-UP CARRIAGE.

when working under extreme changes of load, within the rated capacity up to 150% overload.

The steam consumption was guaranteed not to exceed 19 lb. per i.h.p. per hour for the large engines, and 20 lb. for the smaller engines when operating with an initial steam pressure of 125 lb. per sq. in. and a 26-in. vacuum, developing normal horse power, and at one-half load not to exceed 20½ lb. for the larger engines, and 21½ lb. for the smaller engines. A test was carried out which showed that these conditions were fulfilled. The engines ran very smoothly, and took care of great changes of load with remarkable ease, the speed being regulated to such a degree that three engines could work together in parallel.

The generators were equal to all demands. It was guaranteed that after 24 hours' run at the rated capacity they would have a temperature rise of not more than 35° cent., on the armature, field-coils, and commutator, above that of the air surrounding the machine.

With 150% of full load for 2 hours, the temperature rise of any part of the machine was not to exceed 55° cent. The machines were guaranteed to carry 50% overload for 2 hours without injurious sparking, and 100% overload momentarily without flashing over or injurious sparking. The efficiency was guaranteed to be 91% at full load, and slightly less above and below this. In the larger machines, ½% better efficiency was guaranteed.

In order to measure the current passing to the different parts of the works, the mains were metered at the switch-board. When it was found that the boiler capacity was insufficient for the demand made upon it by the tunnels, a supplementary supply of electricity was contracted for with the Edison Company at Manhattan, and with the Pennsylvania Railroad Company at Long Island City. The Edison supply, being direct current of the same voltage, was coupled up direct to the switch-board, but that at Long Island, being 11 000-volt, 3-phase, alternating, it was necessary to install rotary converters. Two standard Westinghouse 300-kw. converters were selected, wound for direct current at 250 volts, and fitted with 8 poles suitable for a frequency of 3 000 alternations per min., or 25 cycles per sec., at the normal speed of 375 rev. per min. The collector rings were connected for 3-phase current at approximately 153 volts. The rotary converters were started up by an alternating-current motor on the shaft.

The normal full-load rating of the machines was a direct current

of 1 200 amperes at 250 volts. The power factor of the alternating current being 100%, the machines had an efficiency of 90% at one-quarter load up to 94% at full load, and could stand 50% overload for 1 hour with no serious sparking.

The 11 000-volt alternating current, before going to the rotary converters, was transformed to 155 volts low-tension by three oil-insulated self-cooling transformers of 225 kw. each. The combined efficiency was 91.5% at full load; this included the rotary converters and the transformers. When these converters were installed at Long Island City, one of the 160-kw. steam-generating sets was taken over to Manhattan.

The distribution of the electric power consisted of two sets of underground, three-conductor cables from the Pennsylvania Railroad power-house at Long Island City to the Contractors' power-house, for carrying 11 000-volt, 3-phase, alternating current. Here the current was distributed to two sets of step-down transformers, one at 400 volts, for two 600-h.p. motors, driving the air compressors, and the other at 155 volts for the two 300-kw. rotary converters feeding motors and lights at 250 volts, direct current.

Two 160-kw., 250-volt, steam-driven generators were operated in parallel with the converters.

A transmission line, 2 000 ft. long, was run to the East Avenue shaft, with a capacity of 115 kw. at 90% efficiency, consisting of two pairs of 500 000-cir. mils for power and two pairs of No. 000 B. & S. for lights. The electric-light mains in the tunnels at each site were of No. 1 B. & S. At the Manhattan side, the electric power was obtained from the steam-driven generators, three of 75 kw. and one of 160 kw. at 250 volts, with a stand-by connection from the Edison Company at 250 volts.

Shields.—There were two types of shields, namely, the heavy under-river type, Plate C (also shown by Plate LXXXV and Fig. 1, Plate LXXXVI, of the paper by Messrs. Brace, Mason, and Woodard*), and the lighter type, Fig. 13, for the land section driven from the East Avenue shaft under the Long Island Railroad depot. The under-river type was designed by E. W. Moir, M. Am. Soc. C. E., Vice-President of the contractors, and was very like the Blackwall tunnel shield, also designed by him. The principal point of difference between these shields and others used around New York is their massiveness. The

* *Proceedings*, Am. Soc. C. E., for October, 1909.

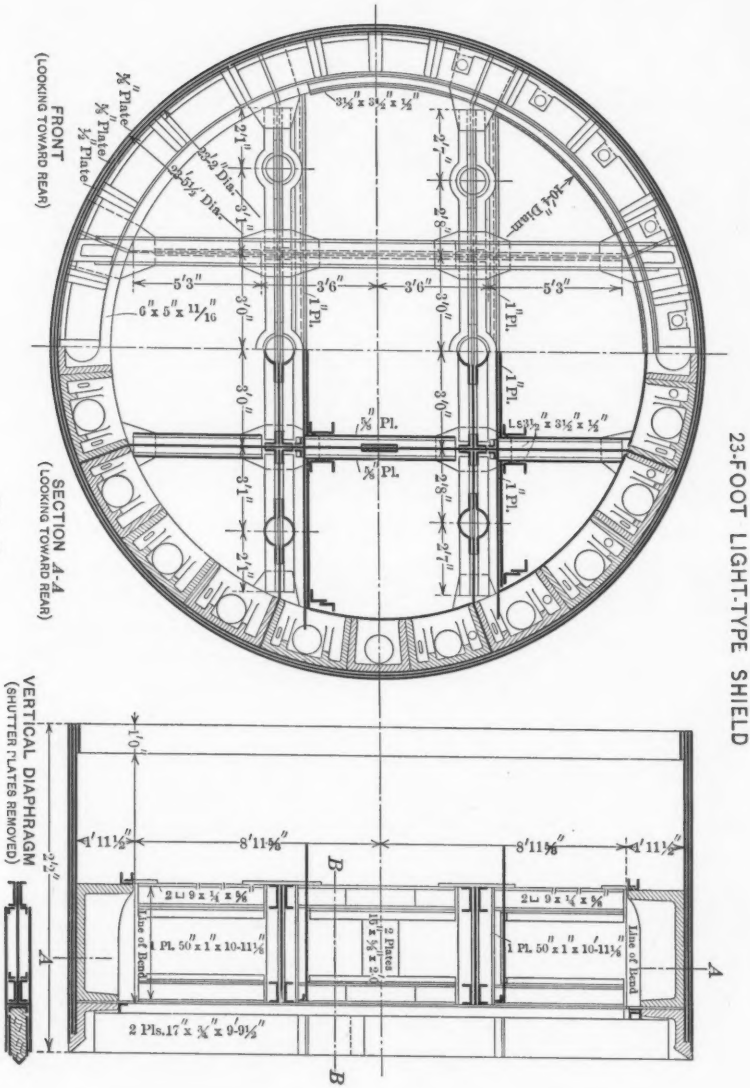

23-FOOT LIGHT-TYPE SHIELD
FRONT
(LOOKING TOWARD REAR)
SECTION A-A
(LOOKING TOWARD REAR)
VERTICAL DIAPHRAGM
(SHUTTER PLATES REMOVED)
SECTION B-B
Line of Bond
2 Plates
2 Pls.17" x ⅜" x 9'-9½"

Fig. 13.

cutting edges, as can be seen from Plate C and Fig. 13, are very heavy, and yet, for the work before them, they proved none too heavy, as one of the cutting edges was turned up by being pushed on an almost imperceptible incline of rock, and had to be repaired under air pressure. The total weight of one of these shields, without jacks or erectors or any accessories, was 185 net tons.

Eight subaqueous shields were ordered from the New York Shipbuilding Company's yard at Camden. They were 23 ft. 6½ in. in outside diameter and 18 ft. long around the edge, with the horizontal floors projecting 9 in. in advance of the cutting edge between the vertical diaphragms, and running back to the line of the cutting edge on each side.

The shields were divided into nine pockets by two vertical diaphragms and two horizontal floors. The horizontal floors were made up of two plates, § in. thick, and were non-continuous for a width of 6 ft. 10 in., butting against the vertical diaphragms which were continuous for a width of 6 ft. 10 in.

To give each floor the necessary beam strength, two continuous doubling-plates, ¾ in. thick and 12 in. wide, and an additional stiffener underneath, made of two 1-in. plates, channel, and Z-bar, passed around the front of the vertical diaphragms to form the compression chord of the beam, and, at the same time, a cutting edge for the horizontal floors. Continuous doubling-plates and angles, that is, two §-in. plates, 18 in. wide, and four 4 by 4 by §-in. angles for each floor, passed around the back of the vertical diaphragm to form the tension chord of the beam. The vertical diaphragms consisted of two §-in. plates with a 1¼-in. space between. Distance pieces or strips, 1¼ in. thick, at a pitch both horizontal and vertical not exceeding 18 in., were riveted between the two §-in. plates to unite them and give proper stability under compression.

To give each vertical diaphragm the necessary beam strength, one plate, 1¼ in. thick and 19 in. wide, and two plates ¾ in. thick and 12 in. wide, were riveted in the front to form the compression chord, the tension chord being formed of one plate, 1¼ in. thick and 12 in. wide, two plates, ¾ in. thick and 12 in. wide, and two 4 by 4 by §-in. angles riveted up the back.

There were three skin plates of ¾-in. steel, the outer and middle plates being 17 ft. 6 in. long. The inner plate was 17 ft. 3 in. long. The

FIG. 1.—CONCRETE BULKHEAD AND AIR-LOCKS.

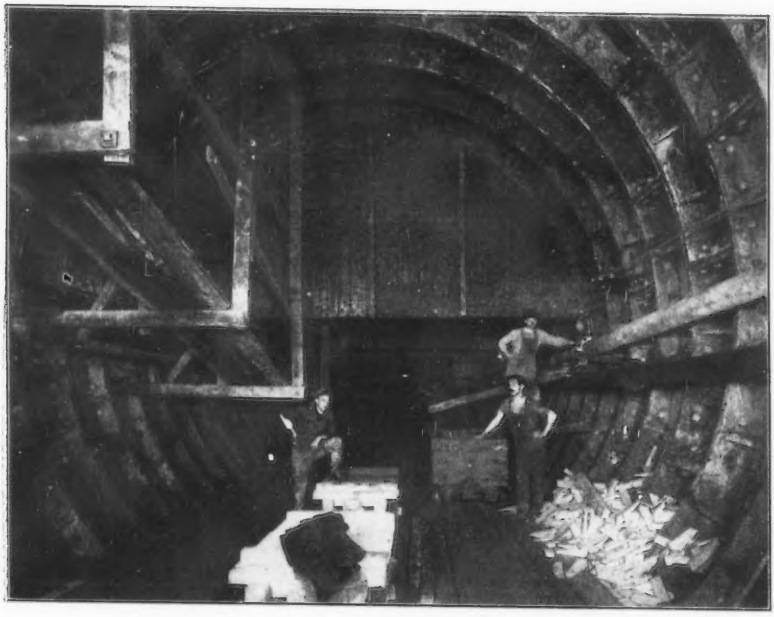


FIG. 2.—SAFETY SCREEN.

skin plates were divided up around the circumference in such a way that the shield could be built for transportation in eight sections, including the hydraulic-jack boxes. The middle and inner skin plates lapped the outer plates by 12 in. and 24 in., respectively.

From the back of the cutting edge, for the full extent of the jack boxes and the stiffening circumferential angle ring, a fourth $\frac{3}{4}$-in. plate was riveted to the skin plates, thus making the skin at this point 3 in. thick.

The inner shell of the jack boxes consisted of two plates, $\frac{5}{8}$ in. thick and 4 ft. $9\frac{1}{4}$ in. long, excepting for a width of 3 ft. $11\frac{1}{4}$ in. in the center top section of the shield where the inner plate was 6 ft. $3\frac{1}{2}$ in. long to form the roof of the safety screen on the back of the shield.

At the rear end of the tail a bead, 18 in. wide and $\frac{1}{2}$ in. thick, was riveted around the inner side of the tail. It had a chamfer 3 in. wide, tapering down to $\frac{1}{4}$ in. thick at each edge. The object of this bead was to permit the shield to be steered without binding on the leading edge of the cast-iron lining. Excepting where the floors and diaphragms divide the jack boxes, they were parted by rolled I-beams, 1 ft. 10 in. by 9 by $\frac{3}{4}$ in., and stiffened by dished plate frames across each box, the rear frame being solid and the center one having a hole 1 ft. 3 in. in diameter for the jack to pass through. All plates were planed on the edges, and all bolt holes and rivet holes were drilled in place wherever possible. The rivets were 1 in. in diameter, and the rivet holes $\frac{1}{16}$ in. greater in diameter. All rivets on the outside were countersunk flush with the skin of the shield, and also on the inside of the tail behind the jack boxes; that is, where the cast-iron tunnel lining is built.

In addition to the two vertical diaphragms, there were two transverse bulkheads, 2 ft. 6 in. apart, completely closing the shields, excepting for openings which were made for doors and muck chutes. For each floor there was a pair of doors, one in each transverse bulkhead, and nine muck chutes pierced both bulkheads, with hinged doors on either end. A ladder, 2 ft. wide, passed from the top to the bottom of the shield on the outside of the right-hand vertical diaphragm, in a vertical iron ladder recess open toward the cutting edge. This was continuous through each floor. Ladders were also provided on the back of the shield for access to the doors.

Two large recesses, about 3 ft. 9 in. wide, open toward the back of the shields, were arranged, one on each side of the shield, between the

horizontal floors, in which were located all the hydraulic controlling valves and the mechanics operating them.

A safety screen, about 4 ft. wide and 7 ft. deep shrouded and surrounded the doors opening from the upper chamber.

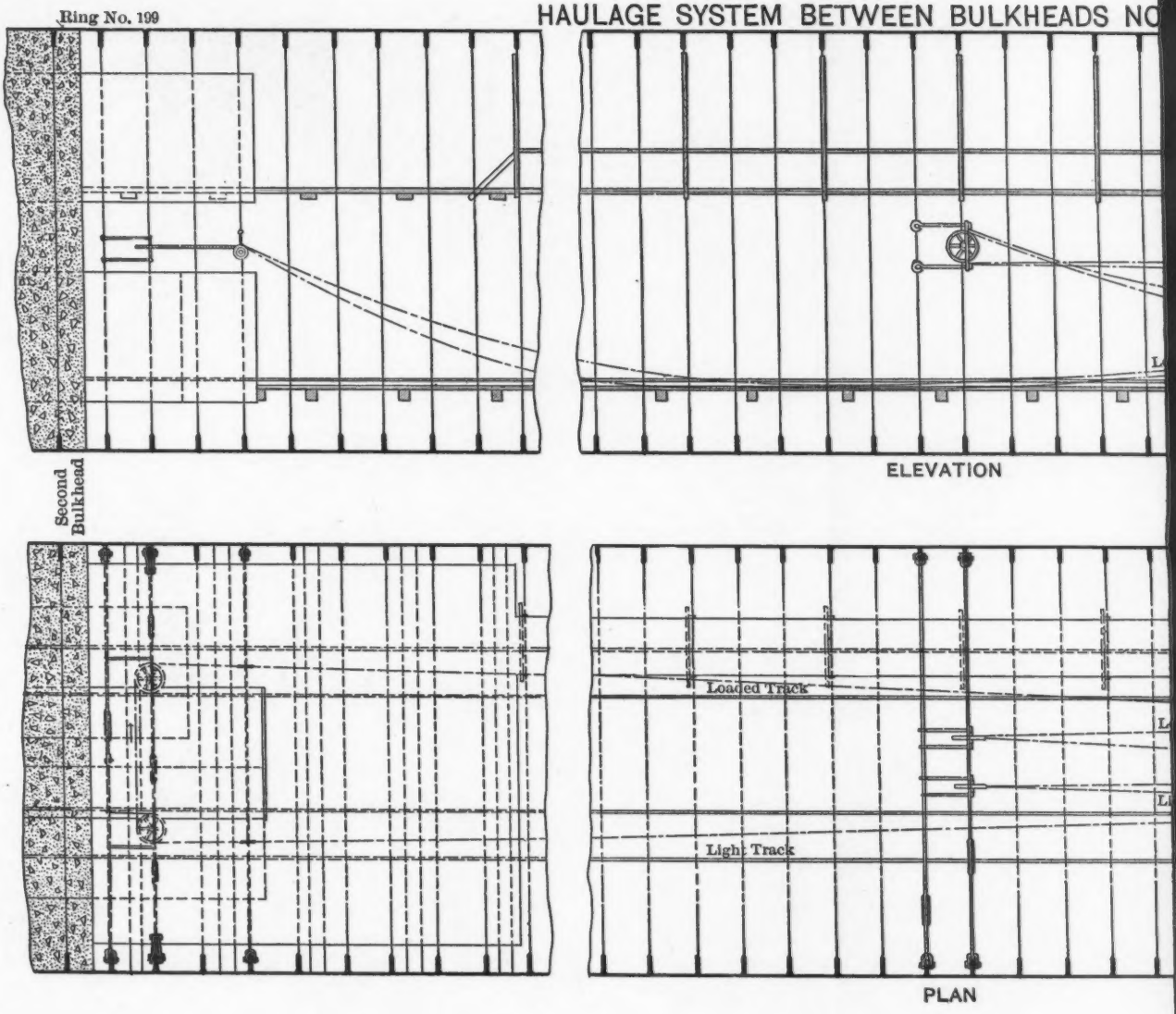
A drop safety curtain, 1 ft. 6 in. deep and ⅜ in. thick, was fixed along the roof of each chamber, and a protecting coffer-dam, 6 in. high, was fixed along the back of each floor, both coffer-dam and curtain being on the same line, 18 in. forward of the forward transverse bulkhead; a continuous passageway opened through the vertical diaphragms for access to each pocket on each floor for the same width.

The cutting edge was of cast steel, divided into segments machined on the radial joints and bolted together with turned and fitted bolts. It was of a shape aptly described by its designer as a lark's foot, and was firmly riveted to the shell plates. The upper part of the cutting edge was especially shaped to receive a temporary fixed hood, and all segments were pierced in the circumferential web with 4-in. holes. The lower half of the cutting edge was pierced radially with tapped test holes, 1½ in. in diameter, which also passed through the shell plates to serve as vertical test holes to find a rock ledge before it was struck by the cutting edge, and, during the tunneling operations, it was found necessary to drill holes through the shell in the roof for the purpose of grouting and also for forcing water to extinguish fires in the timbering and salt hay packing under which the shield had been shoved. Such fires were of frequent occurrence.

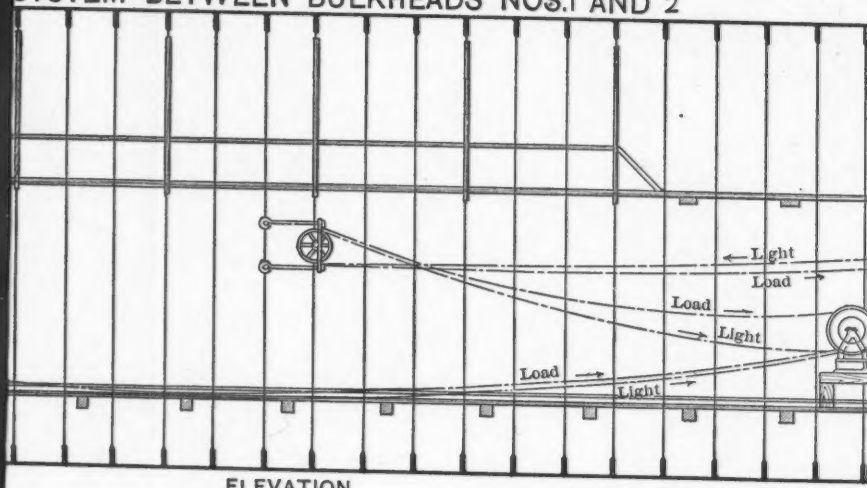
Struts formed of gusset plates, 1 in. thick, with stiffening angles, were also fixed at the back of each jack, not otherwise supported by the floors or diaphragms, to communicate the thrust of the jacks to the cutting edge, and a 9 by 1-in. cover-plate, was riveted inside the inner flange of the cutting edge segments.

Two holes, 8 in. in diameter, for air supply, one 6 in. in diameter for water blow-off, one 6 in. in diameter for foul air exhaust, and one 4 in. in diameter for high-pressure air supply were cut in the bulkheads. All parts of the shield, such as bulkheads, chutes, back ends of jack boxes, etc., between the two transverse bulkheads were made air-tight.

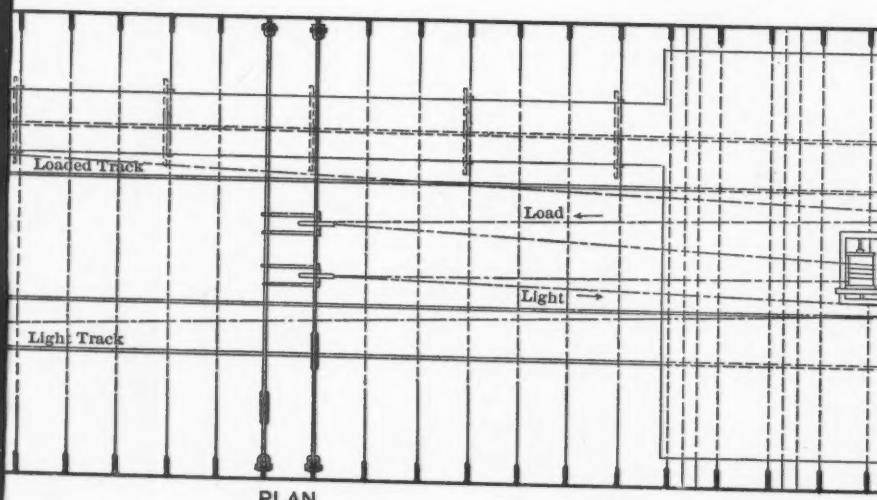
The object of the two transverse bulkheads was to maintain a differential air pressure. By closing the doors of the forward bulkhead, air could be pumped into the shield at a higher pressure than the


Ring No. 199
HAULAGE SYSTEM BETWEEN BULKHEADS NO
Second Bulkhead
ELEVATION
Loaded Track
Light Track
PLAN

SYSTEM BETWEEN BULKHEADS NOS.1 AND 2

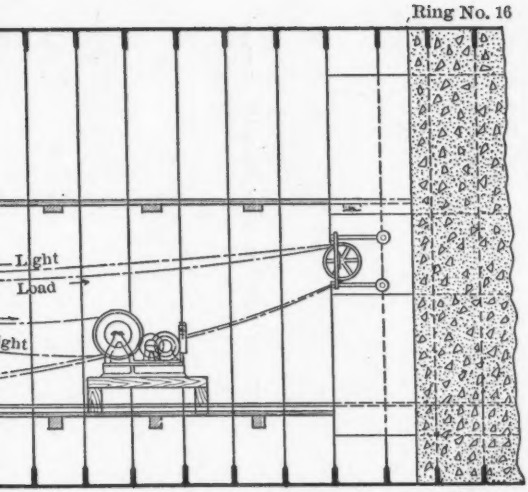


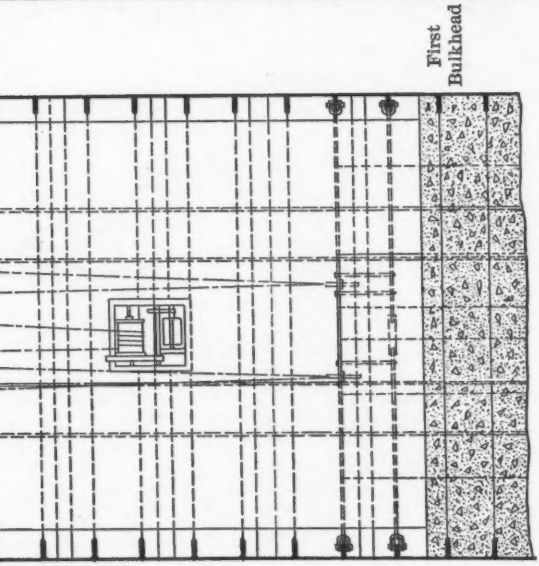
ELEVATION

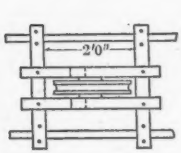


PLAN

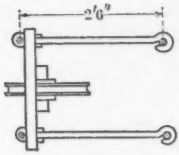
PLATE CVII.
PAPERS, AM. SOC. C. E.
NOVEMBER, 1909.
JAPP ON
CONTRACTORS' PLANT FOR EAST RIVER TUNNELS.



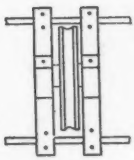




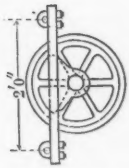
FRONT ELEVATION



SIDE ELEVATION



PLAN



ELEVATION

pressure in the tunnel, and all the excavation could be locked through the muck chutes and the men could lock themselves between the two bulkheads by the use of the air-tight doors.

Such an arrangement would keep as few men as possible under high-pressure air, all the loading of the muck into cars, grouting, building of tunnel lining, and caulking, would be done at a lower pressure than was necessary in the face of the shield. As 37 lb. was the highest pressure ever necessary to keep the water out of the face, this was not done.

The greatest difficulty with this feature, if it had been used, would have occurred in material composed part of rock and part of quicksand in preventing the air pressure in front of the shield from escaping backward along the cavities in the blasted rock into the tunnel proper through the joints of the iron, and between the iron and the tail of the shield, although this would not have been insurmountable, but the small reservoir of air contained in the shield would have caused great variations of pressure in the event of "blows," and flooding would have been frequent in that case.

The benefit of having the two transverse bulkheads is to give the shield an added stiffness which it requires. The smallness of the doors and the muck chutes through these bulkheads handicapped for a time the mucking-out operations, especially in rock, and after it was found that there was no likelihood of this feature being required, the transverse bulkheads in all three bottom pockets were cut out, and the middle bottom pocket was utilized for running the tunnel cars through the shield into the heading beyond on the tunnel track.

To facilitate the passage of drill columns and timber to the upper pockets, part of the center pocket transverse bulkhead was also cut away.

The ladderway inside the shield gave access to all floors in case the differential pressure were used, and was useful for the purpose in any event. The 6-in. coffer-dam along the back of each horizontal floor, 18 in. in front of the doors, prevented sand and débris from being washed against the doors if the shield was flooded, and also kept the passageway clear.

The 18-in. curtain suspended from the under side of each floor, 18 in. in front of the bulkhead, provided an air space for the men if the shield was flooded, when they ducked their heads under this curtain.

The object of the safety screen which shrouded the door of the upper chamber was to prevent the water in the tunnel from rising more than 1 or 2 in. above the bottom of this screen, although the shield proper might be filled to the top with water. This feature was tested once and proved effective when one tunnel was flooded. This, of course, would only be true if the space between the tail of the shield and the cast-iron tunnel was pugged around the upper part of the shield down to the level of the bottom of the safety screen.

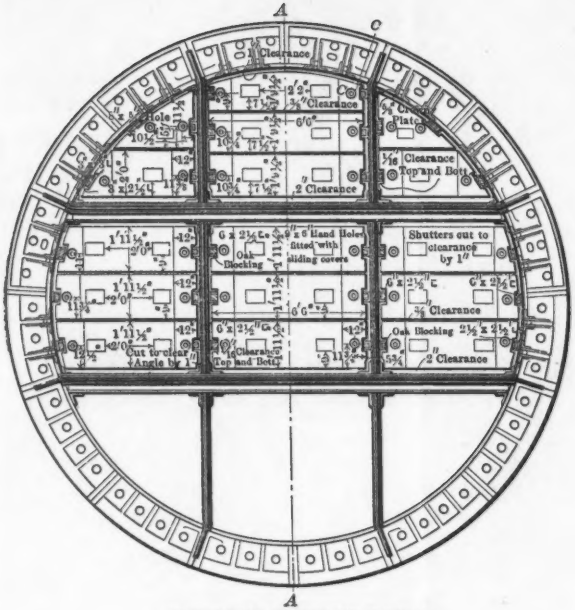
As the conditions obtaining underneath the East River had never been explored by a previous tunnel at the time these shields started work, many and varied conditions and contingencies were provided for in the accessories of the shield which would be unnecessary in future work under this river. At that time the only tunnel under the East River was the gas tunnel at a deep level below the quicksand. The Battery tunnel, it is true, had advanced far from the Manhattan shore, but had been, up to that time, entirely in solid rock. The test borings driven by the Railroad Company's engineers, acting under Mr. Noble, Chief Engineer, disclosed the level of the rock on the lines of the tunnels, but, as they were wash-borings, the nature of the soft material overlying the rock as to its air-holding capacity was somewhat doubtful.

On account of this, sliding shutters, with screw-jacks, similar to those used in the Blackwall tunnel shield in open gravel, were fitted. The details of these are shown on Plate CI. They were used without a hood. Two types of hoods were designed; one was a movable hood, shown on Plate CI,* consisting of steel poling boards lying on the outside of the shield and advanced by means of screws; the other was a fixed hood, shown on Plate CII,† put up in sections. Two types of extension floors were designed and built for the shields; one was a sliding floor moved forward by hydraulic jacks, shown on Plate CI,* the other was a fixed floor.† Both types were carried away by falling rock, and were abandoned. The operation of all these accessories is dealt with in detail in the paper by Messrs. Brace, Mason, and Woodard.

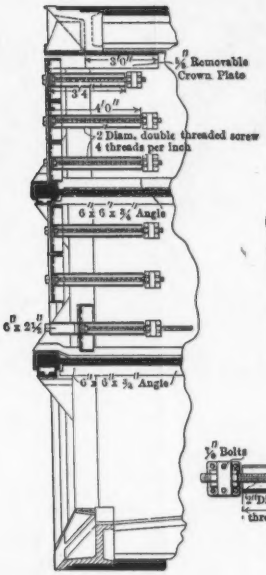
The most satisfactory arrangement, in any type or mixture of types of material found under the river, was the bare shield, with the fixed hood projecting 3 ft. in advance of the cutting edge for about two-fifths of the circumference, and no extension floors except those formed

*Also shown by Fig. 1, Plate LXXXV, of the paper, by Messrs. Brace, Mason, and Woodard, in *Proceedings*, Am. Soc. C. E., for October, 1909.

†Shown by Fig. 2, Plate LXXXV, of above paper.

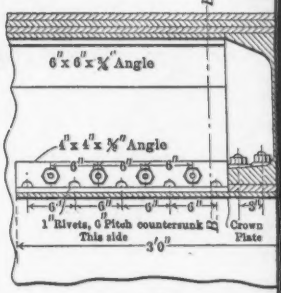


ELEVATION FRONT OF SHIELD
SHOWING SHUTTERS

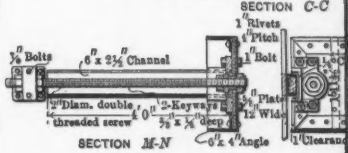


SECTION *A-A*

ARRANGEMENT

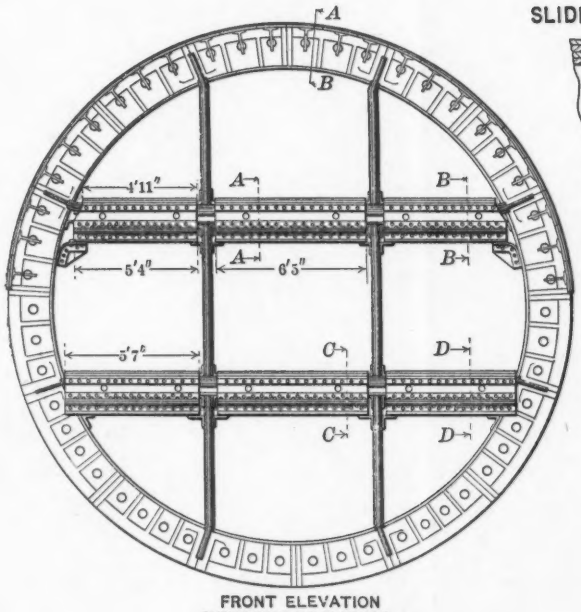


METHOD OF FIXING MOVABLE



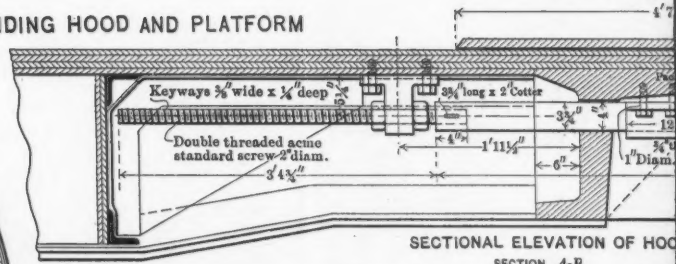
DETAIL OF SHUTTERS IN MIDDLE

Note: The Shutters were designed for use together with the attachments, Cr for Screw-jacks were not placed un



FRONT ELEVATION
SLIDING HOOD AND PLATFORM

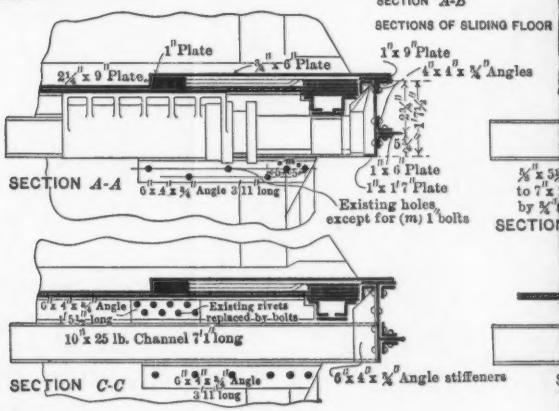
SLIDING HOOD AND PLATFORM

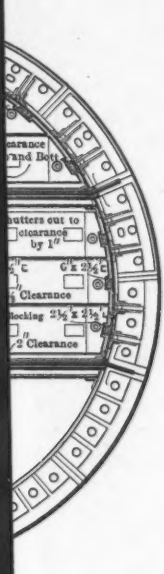


SECTIONAL ELEVATION OF HOO

SECTION *A-B*

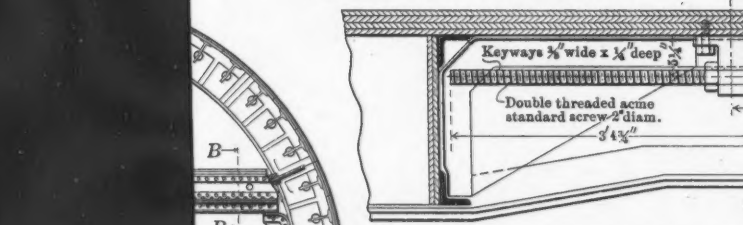
SECTIONS OF SLIDING FLOOR

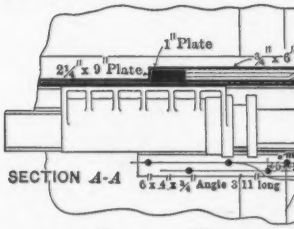




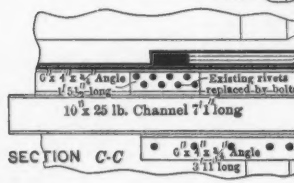
SECTION *A-A*

SLIDING HOOD AND PLATFORM

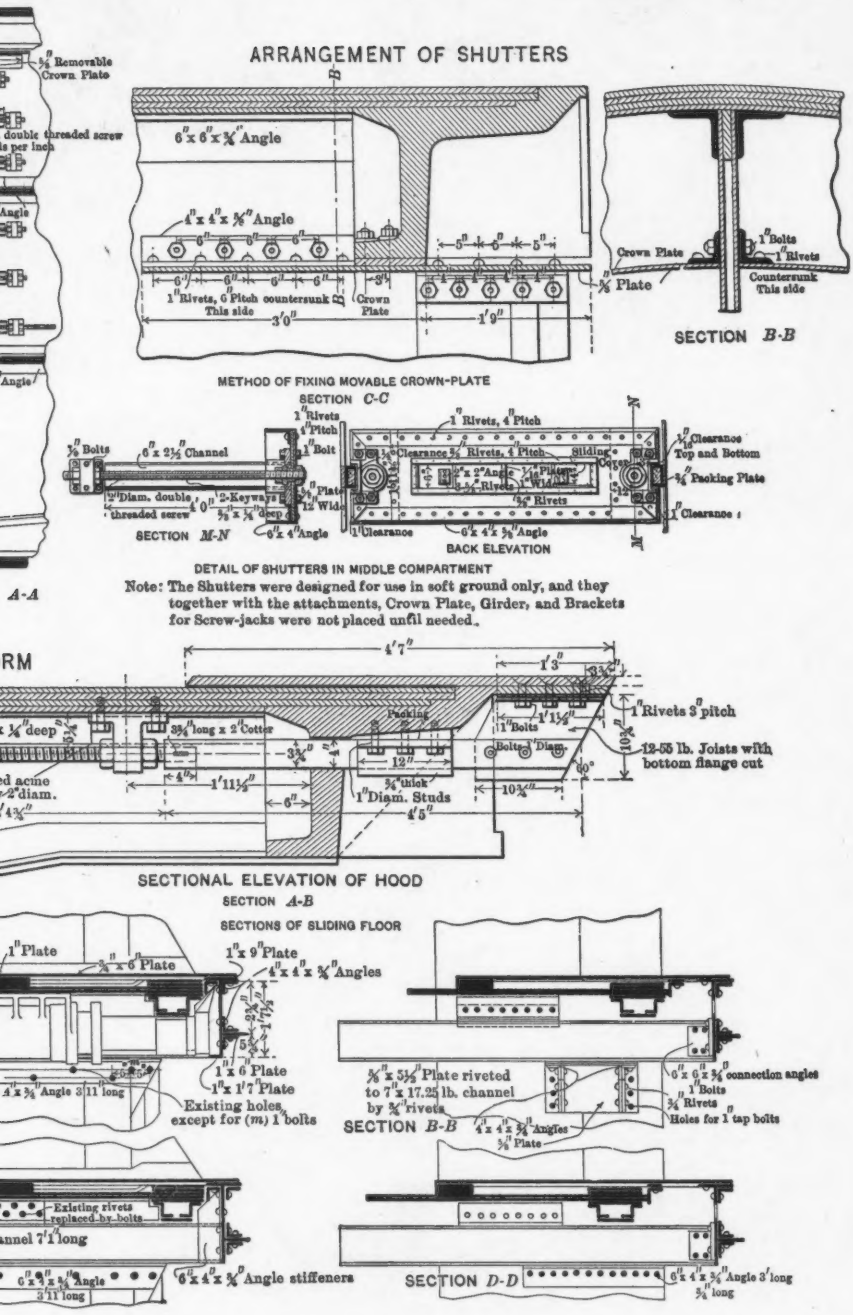




SECTION *A-A*



SECTION *C-C*


ARRANGEMENT OF SHUTTERS
6" x 6" x ¾" Angle
4" x 4" x ⅝" Angle
1" Rivets, 6 Pitch countersunk this side
Crown Plate
3'0"
1'9"
Crown Plate
1" Bolts
1" Rivets
Countersunk This side
⅝" Plate
SECTION B-B
METHOD OF FIXING MOVABLE CROWN-PLATE
SECTION C-C
⅞" Bolts
6" x 2½" Channel
1" Rivets, 4" Pitch
1" Bolt
½" Plate
12" Wld.
½" Diam. double threaded screw
2-Keyways
4'0"
SECTION M-N
6" x 4" Angle
1" Rivets, 4" Pitch
Clearance ⅝" Rivets, 4 Pitch
Sliding Covers
⅝" Rivets
1" Clearance
6" x 4" x ⅝" Angle
BACK ELEVATION
⅛" Clearance Top and Bottom
¾" Packing Plate
1" Clearance
DETAIL OF SHUTTERS IN MIDDLE COMPARTMENT
A-A
Note: The Shutters were designed for use in soft ground only, and they together with the attachments, Crown Plate, Girder, and Brackets for Screw-jacks were not placed until needed.
4'7"
1'3"
1" Rivets 3" pitch
1" Bolts
Packing
12" Diam.
12-55 lb. Joists with bottom flange cut
¾" thick
1" Diam. Studs
4'5"
SECTIONAL ELEVATION OF HOOD
SECTION A-B
SECTIONS OF SLIDING FLOOR
1" Plate
1" x 9" Plate
4" x 4" x ⅝" Angles
1" x 6 Plate
1" x 1'7" Plate
Existing holes except for (m) 1" bolts
⅝" x 5½" Plate riveted to 7" x 17.25 lb. channel by ⅝" rivets
SECTION B-B
6" x 6" x ¾" connection angles
1" Bolts
¾" Rivets
Holes for 1" tap bolts
4" x 4" x ¾" Angles
⅝" Plate
Existing rivets replaced by bolts
6" x 4" x ⅝" Angle stiffeners
SECTION D-D
6" x 4" x ¾" Angle 3' long

by sliding timber extensions, which could readily be replaced if damaged.

These sliding timber floors were built of two 9 by 9-in. timbers in each pocket, decked with 3-in. planking, held down to the floor at the rear end by 8-in. cross-timbers propped from the floor above and back-propped to the bulkhead. These sliding timbers were known by the workmen as cantilevers, and were also used for supporting the poling boards in the roof.

Braces (technically known by the workmen as "guns"), consisting of a 2½-in. pipe, filled with cement grout, sliding inside a 3-in. pipe and gripped with set-screws, were used for bracing the breasting boards. As the shield moved forward the set-screws were adjusted to give enough friction to hold the breasting boards tight while the guns telescoped on themselves. In the same way, the cantilever timbers were adjusted as to friction by wedges between the bearing timbers and the cantilevers.

Removal of Shields.—After the shields met beneath the river, it was necessary to cut them apart, leaving the skins in place, inside which the tunnel lining was built. As the labor of cutting them out had to be performed in compressed air, and represented an enormous amount of work, it was decided to burn them out by electricity.

The current for burning the shields was obtained from the Long Island City power-house, from one of the 300-kw., 250-volt, rotary converters. Occasionally, when burning one shield, only one of the steam-driven 160-kw. sets was used. The circuit for burning was taken from the positive terminal of the generator, and was conducted through a water-resistance consisting of four barrels of submerged iron plates, about 18 by 14 by ¼ in., all four barrels being connected in parallel. This resistance served to reduce the voltage to approximately 45 to 60 volts at the burning tool in the tunnel, and also for regulating the current according to the number of tools working simultaneously. From the water-resistance the current was conveyed to ground-plates submerged in the river, the total area of these plates being about 48 sq. ft. From the negative terminal of the generator, a cable was run to the shields direct, branching off at the shafts, one branch going down Tunnel *D* and the other down Tunnel *B*. A connection was made to Tunnels *A* and *C* from Tunnels *D* and *B* through a small pipe driven between the two tunnels. The cables used were 500 000 cir. mils, and the capacity was about 650 amperes.

The burning tool consisted of a carbon 12 in. long and 1 in. in diameter bolted to a copper rod. This copper rod was connected to the cable leading directly from the generator to a single-pole switch near the shields. The tool was provided with an insulated handle and a shield of asbestos, 12 in. in diameter, for protecting the hands of the workmen from exposure to the light. The men were also protected by an asbestos mask, with dark-colored eye-glasses, and an asbestos apron.

The current consumed for burning the shields varied from 250 to 400 amperes per tool for burning off rivet heads and for light section plates, while from 600 to 800 amperes were required for burning through the floor plates and uprights, the uprights being about 4 in. thick. The best results were obtained with about 40 volts at 600 amperes, the arc being from $\frac{1}{2}$ to $\frac{3}{4}$ in. long.

To cut off 300 rivet heads was considered a fair 8-hour shift's work; 530 rivet heads was the highest record. One man, who was specially expert, cut off one upright in 8 hours, the amount of metal being approximately 4 ft. 6 in. by 4 in.

Several of the workmen cleaning up the tunnel some distance away from the burning were blinded by the light. It takes but a few seconds, looking directly at the arc, to cause one to turn blind temporarily a few hours later, and causes violent headaches.

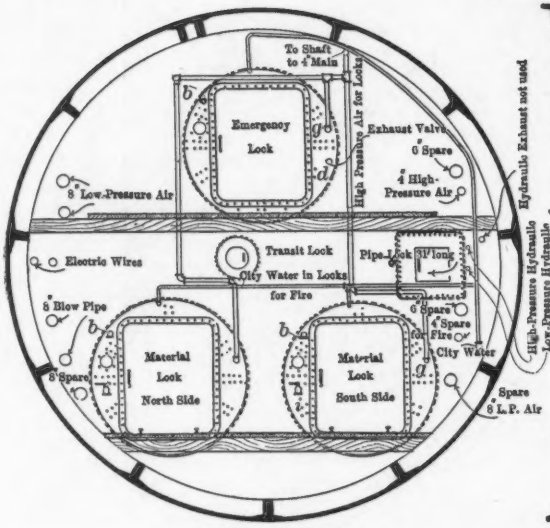
Dense fumes arose from the burning metal, which for the most part were carried off directly by a foul-air blow-out pipe suspended near the arc, but great quantities escaped into the tunnel, and it was often so dense that one could walk along the lighted tunnel with difficulty by the aid of a candle. No specially high CO_2 was recorded on account of this, and no bad cases of "bends" occurred in this foul atmosphere, although the air pressure was 29 lb.

Light-Section Shields.—The shields of a lighter section were designed for the work carried out under air pressure at East Avenue. The first one, which was made from an old London tube shield, was unable to stand the weight of earth above it, on account of being in rock for half the diameter. The large annular space between the outside of the shield and the jagged edges of the rock excavation gave it no reaction to resist the downward pressure of the earth in the top.

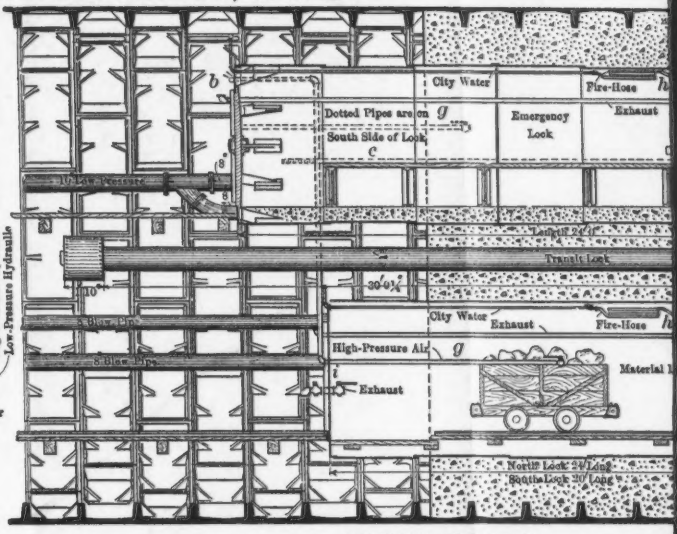
Fig. 13 shows the light-section shield, which was made for the work under compressed air under the Long Island Railroad depot. This was none too strong in the rock section with soft top, but

CONT

LOCKS IN NO. 1 BULKHEAD, TUNNEL *A*

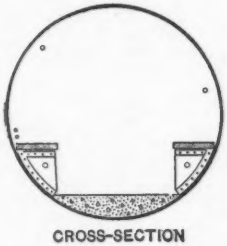


FRONT ELEVATION

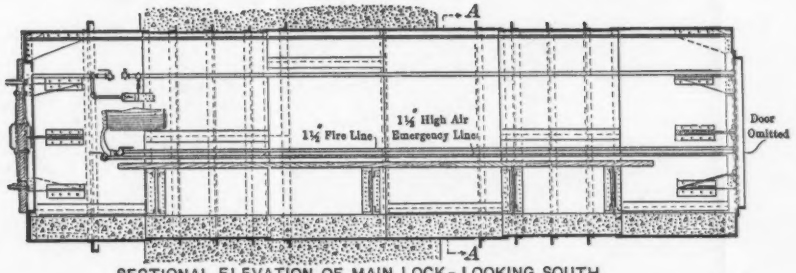


SECTIONAL ELEVATION
ALSO INSIDE OF MATERIAL LOCK

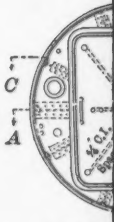
DETAILS OF AIR-LOCKS



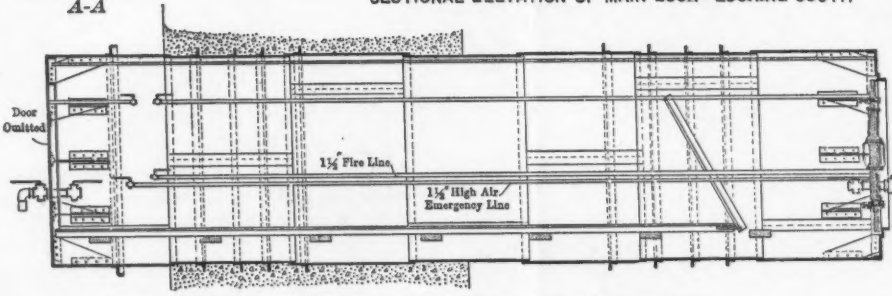
CROSS-SECTION
A-A



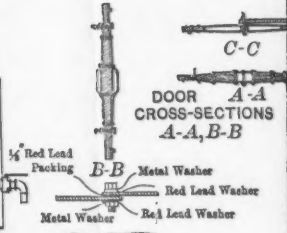
SECTIONAL ELEVATION OF MAIN LOCK - LOOKING SOUTH



DETAIL

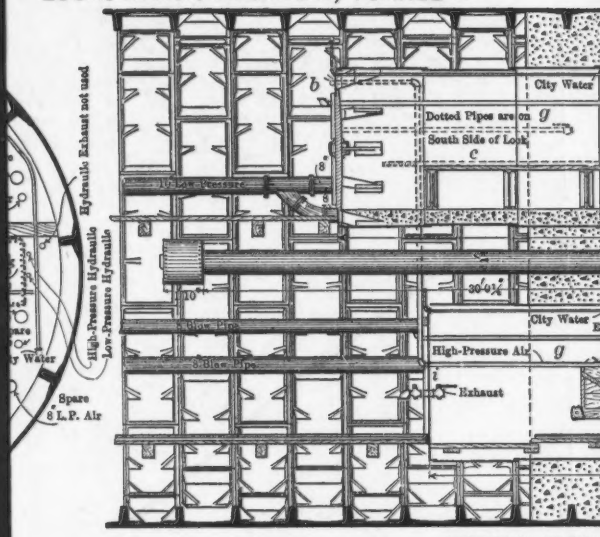


SECTIONAL ELEVATION OF MATERIAL LOCK—LOOKING SOUTH



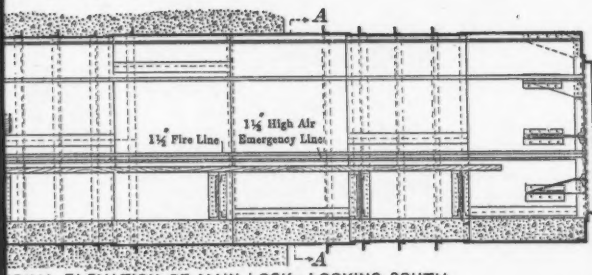
DETAIL-SPLICE OF LOCK PLATES

LOCKS IN NO.1 BULKHEAD, TUNNEL *A*

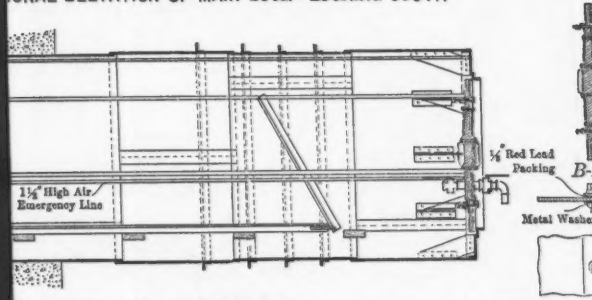


SECTIONAL ELEVA
ALSO INSIDE OF MATER

LS OF AIR-LOCKS



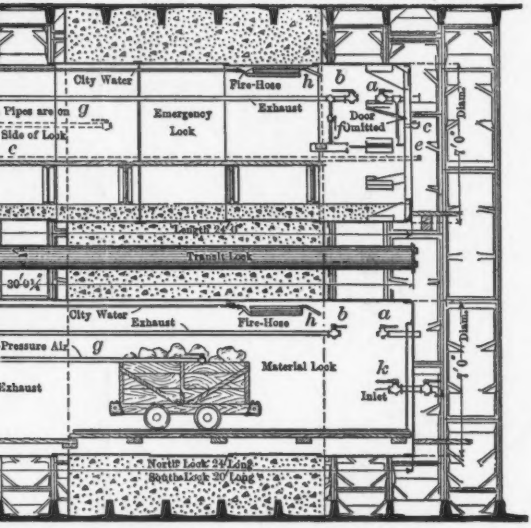
IONAL ELEVATION OF MAIN LOCK- LOOKING SOUTH



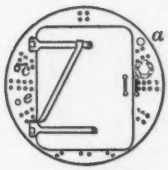
MATERIAL LOCK—LOOKING SOUTH

DETAIL-SPLICE

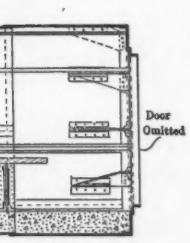
A

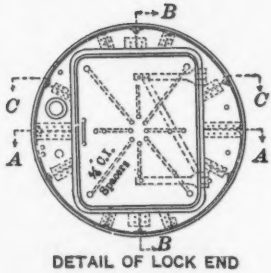


CTIONAL ELEVATION
NSIDE OF MATERIAL LOCK

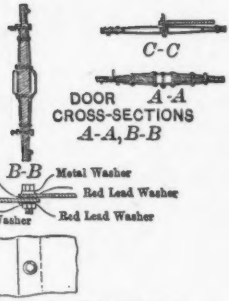


END VIEW OF EMERGENCY LOCK





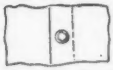
DETAIL OF LOCK END



DOOR CROSS-SECTIONS A-A, B-B

DETAIL OF HINGES

1/8" Red Lead Packing



DETAIL-SPLICE OF LOCK PLATES

answered its purpose quite successfully. It will be noticed that the back of the shield is quite open, although a light stiffened diaphragm was available for bolting on it if flowing material were met. This shield was built and dismantled in three tunnels, namely, *D*, *C*, and *A*, East Avenue.

Each shield was fitted with 27 hydraulic jacks (Plate CII) 9 in. in diameter with 3-ft. stroke for the subaqueous shields, and 8 in. in diameter for the light-section shields. The head of the jack was offset 10½ in. from the center, and was beveled off so that the pressure would be transmitted to the cast-iron tunnel lining on the circumferential web, instead of on the circumferential flange. The cylinders of the jacks were of solid forged steel.

Each jack was tested to a pressure of 9 000 lb. per sq. in. with the ram in position, but the head of the ram, not being strong enough to withstand an offset pressure of this amount, was held centrally. They were machined so that when the cylinder was up-ended and the ram inserted it fell gently of its own weight into the cylinder, all the waterways being open.

In operating the first shields, great trouble was experienced in keeping the draw-back spindles tight, and it was found that the U-leathers were cutting very quickly, partly due to the poor quality of the leather and partly to the fact that the brass backing ring was too loose on the draw-back spindle and allowed the pressure to force part of the leather into the annular space between the backing ring and the spindle. Imported U-leathers were then used, and they lasted somewhat longer. Woodite was very little better, and ultimately Dermatine, an imported substitute, was found to be most successful. Dermatine seems to be built up with rubber and silk. Previous to its use, pieces of the damaged U-leathers got into the waterways, and very soon choked the passage of the water.

To help the lubrication, 5 lb. of soft soap were put in the hydraulic water tank for each shove made, and in the winter the water was heated by steam to prevent freezing.

Much trouble was also caused at first by very fine quicksand being sucked through the small opening in the cast-iron plug into the internal bore of the ram. This was obviated by putting a small non-return valve in the opening, which allowed any leakage to escape but prevented any dirty water from being sucked into the draw-back. Owing to the large

offset pressure on the ram head and the softness of the steel, both of the cylinder and of the ram, it was found that they were cutting into each other and ultimately cutting so deeply as to defy all movement. It was found, on taking out the first damaged ram, that a piece of steel as big as a ¾-in. rivet head had been torn out of the ram and smaller pieces from the cylinder. These portions were analyzed, with the following results:

	Ram of shield jack.	Cylinder of shield jack.
Carbon	0.41%	0.47%
Manganese	0.57%	0.50%
Phosphorus	0.02%	0.016%
Silicon	0.009%	0.019%
Sulphur	0.045%	0.038%

Mr. A. W. Gibbs, General Superintendent of Motive Power of the Pennsylvania Railroad Company, whose chemist, C. B. Dudley, M. Am. Soc. C. E., made the analysis, suggested that the silicon should have been about 0.20%, and, as there were no shock strains, a steel considerably higher in carbon, viz., 0.60 to 0.70%, would give much more successful results.

The seizing of these rams was quite a serious matter, and, in order to get them repaired, it was necessary that the ram and the cylinder in which it was jammed be pulled out of the shield. To do this, one plate was left out of the ring opposite the damaged jack, and, on shoving 2 ft., the jack, being lashed to the tunnel lining, was left behind and removed, and the plate was pushed into place.

The method adopted for repairing was to machine the damaged part of the ram in the lathe down to 8 in. in diameter for a distance varying from 9 to 18 in., depending on the extent of the damage, and bush this with a hard gun-metal sleeve. When this was done, no further trouble was experienced, and, after the work was finished, the bushed rams showed no signs of seizing.

As the bush in most cases came outside the gland of the ram, on a full shove it was necessary to make it water-tight, so that the hydraulic pressure could not get under the bush and burst it when it came outside the gland.

The jacks were coupled up in pairs to the controlling valves (Fig. 14) in the valve chambers. Each valve consisted of three steel

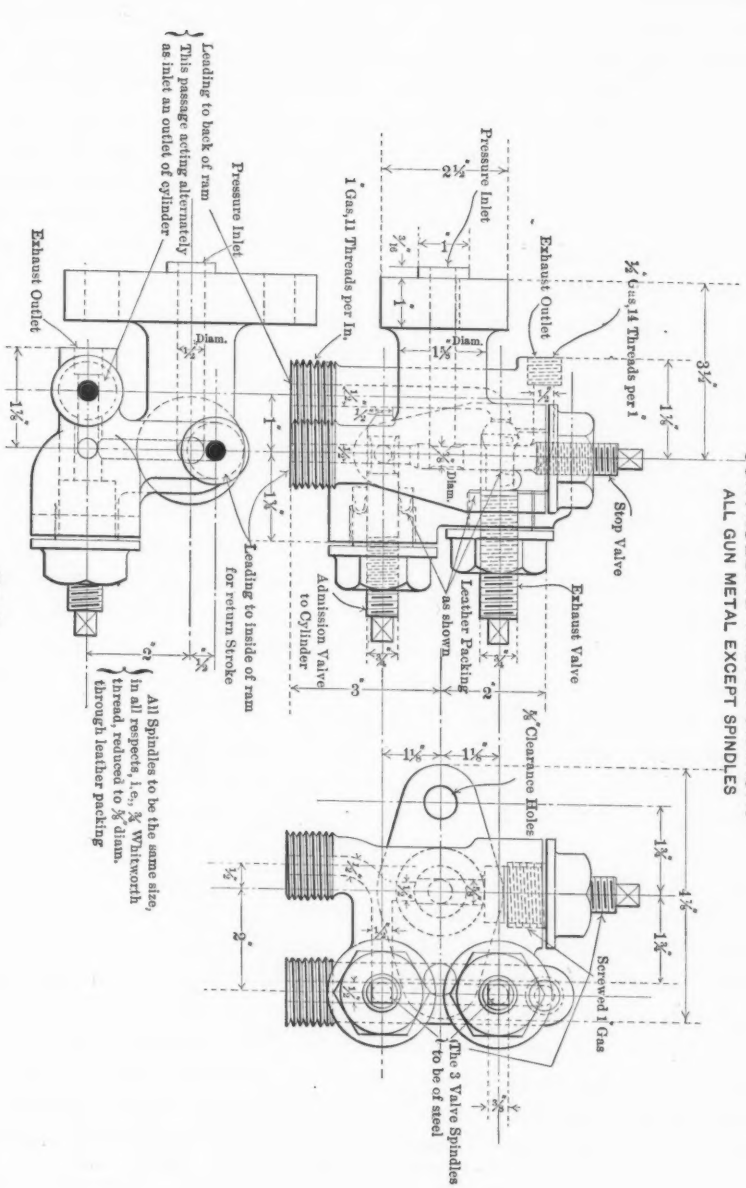

CONTROLLING VALVE FOR JACKS
ALL GUN METAL EXCEPT SPINDLES
Pressure Inlet
Exhaust Outlet
Stop Valve
Exhaust Valve
Admission Valve to Cylinder
Leather Packing as shown
Clearance Holes
Screwed 1" Gas
The 3 Valve Spindles to be of steel
Leading to back of ram
This passage acting alternately as inlet an outlet of cylinder
Leading to inside of ram for return Stroke
All Spindles to be the same size, in all respects, i.e., ⅞ Whitworth thread, reduced to ⅝" diam. through leather packing

FIG. 14.

conically-ended spindles seating on the gun-metal body of the valve, which had four connections. One spindle controlled the high-pressure water from the main, while, of the remaining two, one controlled the exhaust and the other the admission of water to the rear end of the ram.

The draw-back waterway was under pressure as soon as the spindle controlling the pressure was opened, so that, assuming that all was in readiness for shoving, the spindle controlling the pressure being opened, and the exhaust shut, the operator, on opening the third spindle, allowed water to pass to the back of two rams which moved forward, the water at the back of the draw-backs circulating to the main.

Depending on the direction in which it was desired to guide the shield to maintain the engineer's lines, the jacks, in pairs on either side of the shield, were opened up, and also in pairs, top or bottom of the shield, depending on the grade required.

In order to stop all the jacks simultaneously, to prevent overshoving, it was usual to open a by-pass on the high-pressure main; this reduced the pressure sufficiently to stop the shield. The spindles controlling the admission of the pressure to the back end of the jacks were then shut for each pair, and, after telephoning to the power-house to slow down the hydraulic engine, the by-pass was closed and the rams were drawn back in pairs by opening the exhaust spindle. A pressure of 2 000 lb. was sufficient to draw back the rams, and if more was required it was a sign that the rams were scoring, or that the draw-back leather required renewing. If they failed to draw back, they were pushed home with a small portable hydraulic jack. Copper pipes, ½ in. in diameter, were used for coupling the valves to the jacks and ¼ in. in diameter for the draw-back; and all the waterways in the jacks and draw-back spindles were ½ in. in diameter.

Erectors.—Two hydraulic erectors (Fig. 15*) were fixed on the back of each subaqueous shield. On the light-type shield the erector was fixed on the traveling stage. These were used for building the cast-iron tunnel lining. They consisted of an arm, which could be extended, and at the same time rotated. The extension was accomplished by a hydraulic cylinder, 4¼ in. in diameter, with a piston of that diameter attached to a ram 3 in. in diameter passing through a gland at one end of the cylinder, the piston being fitted with a leather

*Shown also by Fig. 1, Plate LXXXVI, of the paper by Messrs. Brace, Mason, and Woodard, *Proceedings*, Am. Soc. C. E., for October, 1909.

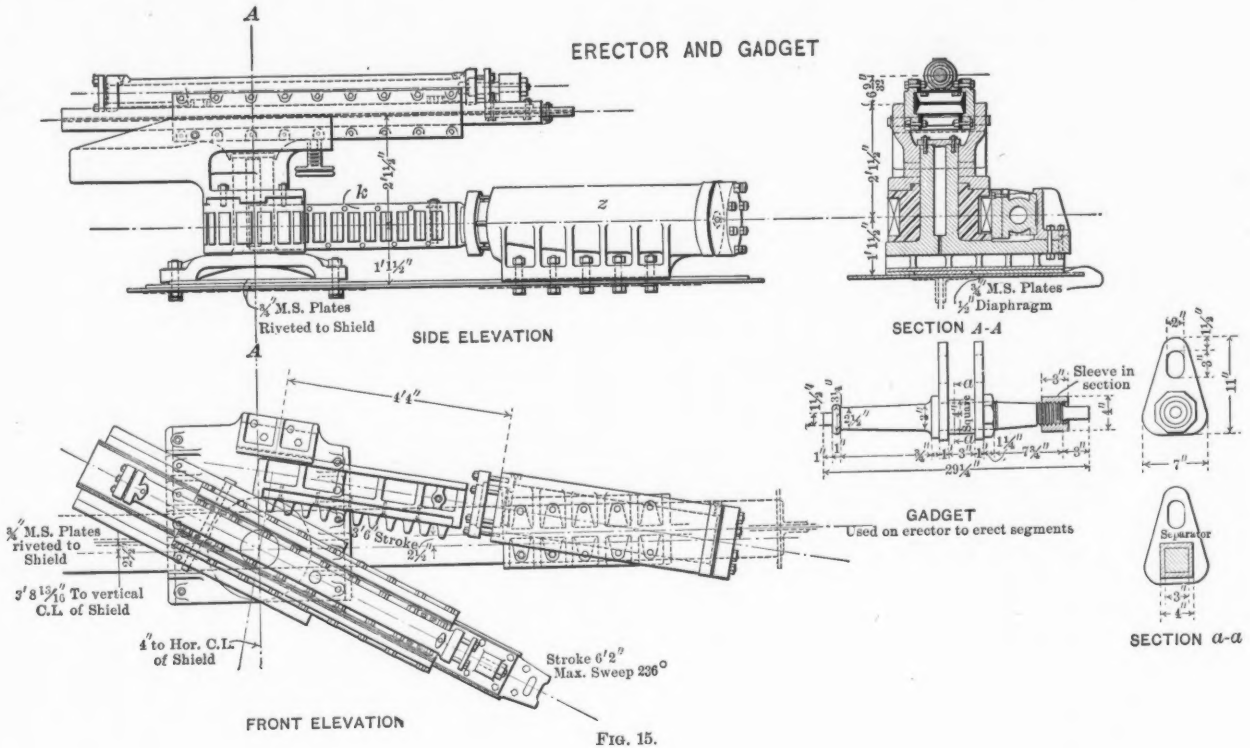

ERECTOR AND GADGET
A
k
z
2'1½"
1'11¼"
¾" M.S. Plates
Riveted to Shield
SIDE ELEVATION
SECTION A-A
¾" M.S. Plates
½" Diaphragm
A
4'4"
¾" M.S. Plates
riveted to
Shield
3'8 15/16" To vertical
C.L. of Shield
4" to Hor. C.L.
of Shield
3'6 Stroke
Stroke 6'2"
Max. Sweep 236°
FRONT ELEVATION
GADGET
Used on erector to erect segments
Sleeve in
section
Separator
SECTION a-a

FIG. 15.

which acted only on the backward stroke, or the equivalent of a ram 3 in. in diameter acting in either direction.

Bolted to the plunger by a cast-iron bracket was an I-beam, 9 ft. 6 in. long and 11¼ by 5 in., the web extending 7 in. beyond the flanges at one end. This extension was reinforced by two ½-in. plates and pierced with an oblong hole, 5 by 1¾ in. The I-beam slides, with a stroke of 6 ft. 0 in., in a guide-box which is held in one end of a rotating casting by a 1½-in. pin on which it hinges, and on the other end by two spherical washers with elliptical holes on the end of an adjustable spindle screwed into the slide-box. An internal-screwed spindle holds the outer washer to jam the slide-box at the required angle.

This allows the erector to be adjusted if the shield has been shoved too far, or if working with an excessive lead.

The cast-iron rotating piece is bolted to a cast-steel pinion having a pitch circle of 20 in. The arm with the pinion is rotated by a rack on the 8½-in. plunger of the 11½-in. slewing cylinder, which has a piston 11½ in. in diameter. The piston has a leather which acts on the upward stroke only, or the equivalent of a ram 8½ in. in diameter working either way.

The erectors are controlled by a two-way slide-valve, shown on Plate CIII. The pressure enters on the upper side of the slide and passes directly to the under side of the slewing cylinder. If the slide-valve is moved over so that the upper end of the cylinder communicates with the exhaust port, the ram moves upward and the erector rises, but, if the slide is set so that the upper end of the cylinder communicates with the pressure, the ram moves downward, the water on the under side of the piston circulating through the valve-box to the main.

With a similar valve, the push-and-pull cylinder on the rotating arm was controlled. The connection to this cylinder was made by two flexible, armored-hose pipes, leaving the erector free to rotate. Water was admitted to the erectors through a safety, non-return, stop-valve, shown on Plate CIII, so that, if the pressure failed, the erectors would not fall. These erectors were designed to handle segments weighing 1 ton each, with a pressure of 1 000 lb. per sq. in. To accelerate their movement, the pressure was ultimately raised to 1 200 lb. This was simpler than increasing the diameter of the openings in the valves and pipes. The erector arm was attached to the tunnel segments by a gadget (Fig. 15).

PLAN

Fig. 17. END VIEW AND ELEVATION SHOWING PIN AND SLOT ARRANGEMENT

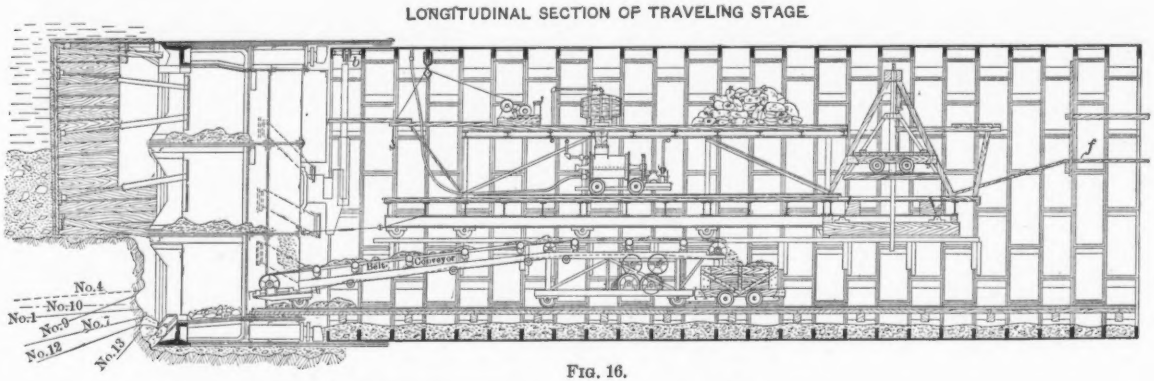

LONGITUDINAL SECTION OF TRAVELING STAGE
No.4
No.1
No.10
No.9
No.7
No.12
No.13
f

FIG. 16.

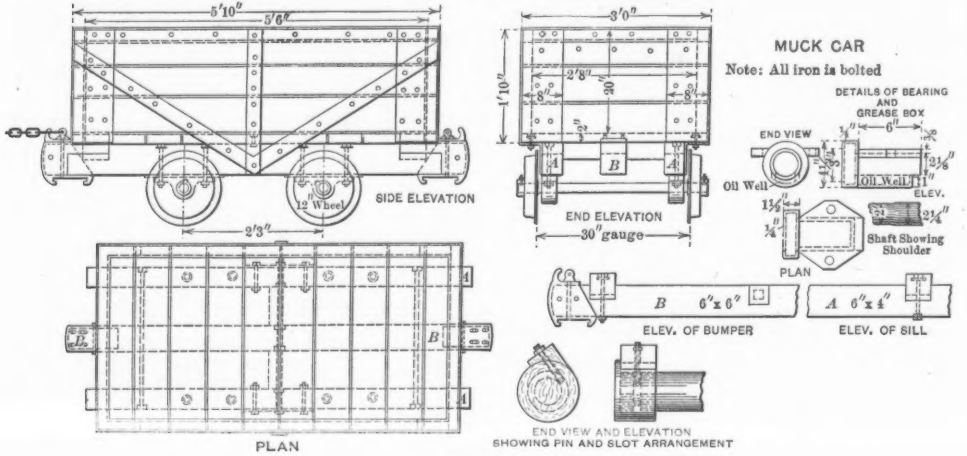

5'10"
5'6"
12 Wheel
SIDE ELEVATION
2'3"
B
A
PLAN
3'0"
2'8"
1'10"
30
8"
END ELEVATION
30" gauge
MUCK CAR
Note: All iron is bolted
DETAILS OF BEARING AND GREASE BOX
END VIEW
6"
Oil Well
ELEV.
Shaft Showing Shoulder
PLAN
B 6"x 6"
ELEV. OF BUMPER
A 6"x 4"
ELEV. OF SILL
END VIEW AND ELEVATION
SHOWING PIN AND SLOT ARRANGEMENT

FIG. 17.

Behind each shield a traveling platform (Fig. 16), 46 ft. long and 20 ft. 6 in. wide, on twelve rollers, 12 in. in diameter and 12 in. long, running on 60-lb. rails, supported by steel brackets bolted on every second ring to the circumferential flanges of the tunnel lining, was attached by two union screws to each shield, and was pulled along by it during each shove. The stage was double-decked. On the upper deck the cement and sand for grouting were stored, and a cantilever extension in front was used for bolting up the iron; on the lower deck the grout-mixing machines were fixed. It was originally intended to use these stages for taking care of the excavation, so that it could be handled from two levels, but the output was never large enough to require this, as casting the excavation directly into cars on the floor of the tunnel sufficed.

A hydraulic elevator at the rear end of the stage, for raising cement and sand, and an air-winch at the forward end, for hoisting lumps of rock into the cars, were the only mechanical attachments, excepting in the case of Tunnel *A*, Manhattan, where a belt conveyor was tried, as shown on Fig. 16, under the stage, and by Fig. 2, Plate XCVIII. The rails on which it traveled were in 6-ft. lengths, spanning two brackets and bent down at the rear end to catch the brackets.

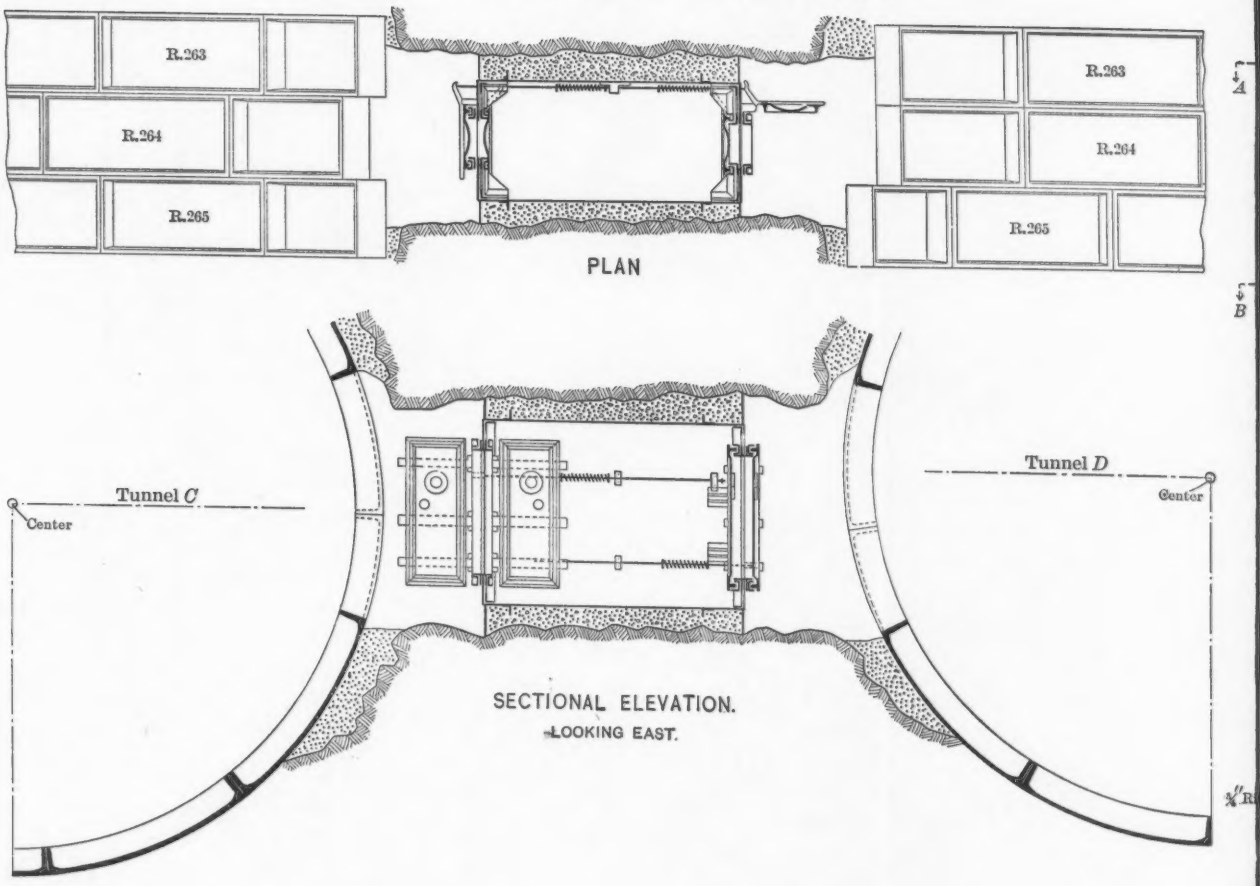
Rock Drills.—The standard Ingersoll, *E* 52, 3¼-in. rock drill was adopted, after testing various makes. They were found to use less air than other makes and to stand up to the work equally well, if not better. They had exceptionally hard service on account of the seamy nature of the rock. They were generally mounted on Standard drill columns, set up in the pockets of the shield, except when advance headings were being driven.

In addition, for small trimming work and for breaking up lumps of rock, little Jap drills, 1¼ in. in diameter with 4-in. stroke, and *M V* drills, $1\frac{11}{16}$ in. in diameter with 2¼-in. stroke, made by the same company, were used.

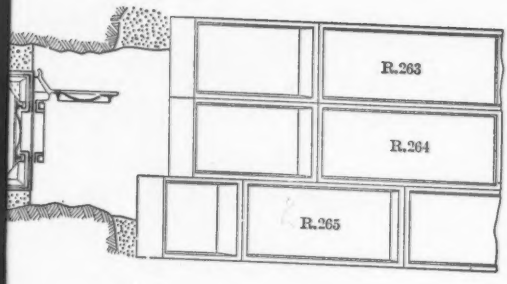
For rust and lead caulking, Boyer and Keller hammers, $1\frac{1}{16}$ and $\frac{15}{16}$ in. in diameter, made by the Chicago Pneumatic Company, giving 2 800 blows per minute, were found most effective; and, for drilling junction segments, etc., Little Giant rotary pneumatic drills were used.

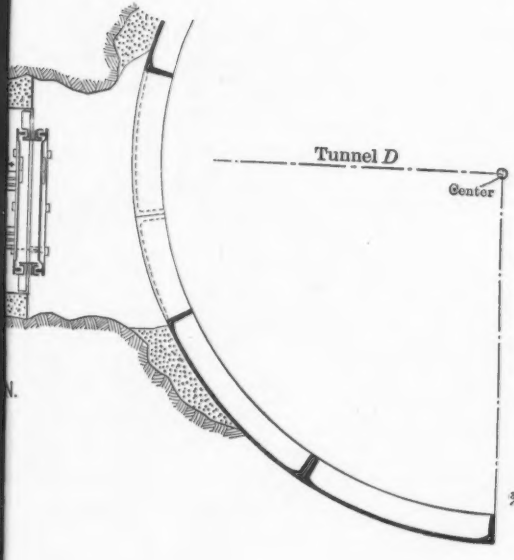
Grouting Pans.—The grouting pans, Fig. 1, Plate CIV, were of the Greathead type, made mostly by the Cockburn Barrow Company, driven by 3-cylinder Ingersoll air-motors, the air-motor being geared to the

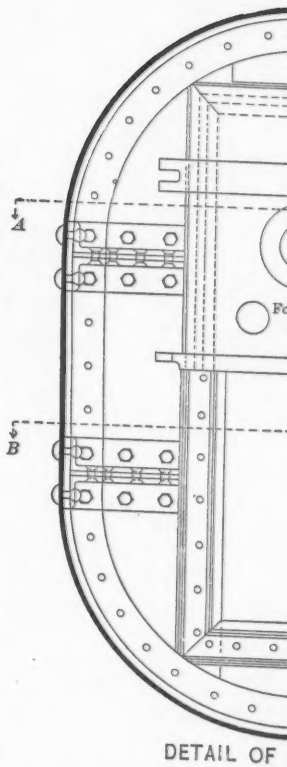
DETAILS OF CROSS-PASSAGE AIR-LOCK *C–D*



AIR-LOCK *C-D*







DETAIL OF

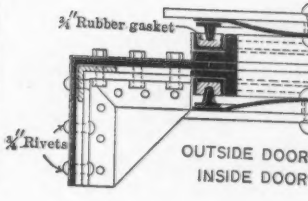
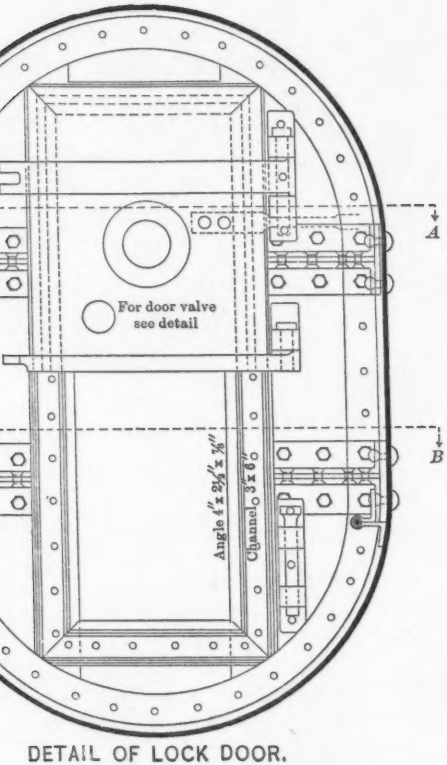
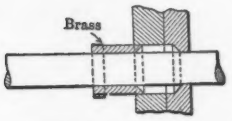


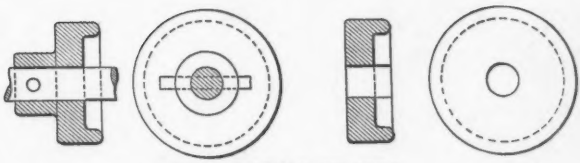
PLATE CX.
PAPERS, AM. SOC. C. E.
NOVEMBER, 1909.
JAPP ON
CONTRACTORS' PLANT FOR EAST RIVER TUNNELS.



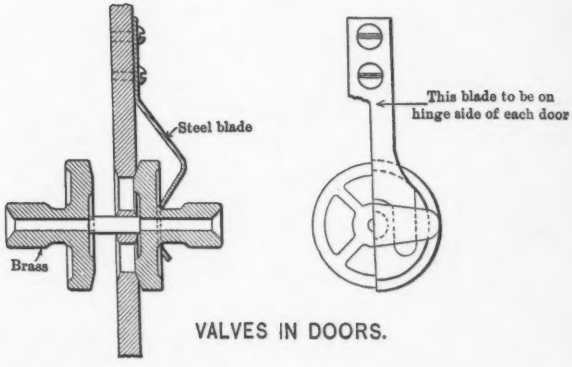
DETAIL OF LOCK DOOR.



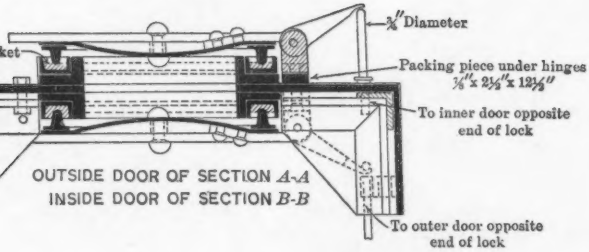
STUFFING-BOX.
PASSING ROD THROUGH DOORS.



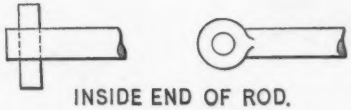
STEEL SPRING PLATES.



VALVES IN DOORS.



OUTSIDE DOOR OF SECTION *A-A*
INSIDE DOOR OF SECTION *B-B*



INSIDE END OF ROD.

central spindle which rotated the beaters or mixing blades. The air-motor was kept revolving during grouting operations, and, after half filling the pan with water, the mixture of sand and cement was dumped into the pan, and, the small hinged door on the top being pulled upward, air pressure was turned on, which held the door tight and the charge was then ejected through a flexible, armored-hose pipe coupled up to a grout hole in the tunnel lining, there being one of these in every segment. After the pan was emptied, the air pressure was shut off and an exhaust valve was opened. This emptied the pan of pressure and allowed the door to open inward, ready for a fresh charge.

The grout pans were located on the traveling stage, and were mounted on wheels of 30-in. gauge, so that they could be hauled along the tunnel tracks, and taken on top for repairs. To save time in charging them with water, a barrel with a 3-in. cock was fixed on the upper platform, so that water could be collecting there while the pan was discharging.

Muck Cars.—The muck cars, Fig. 17, were 24-cu. ft. wooden cars. The wheels were loose on the axles, and the axles loose in the pedestals. This made it very easy to get them around sharp curves, but the final result was not good, as the bushing of the wheels wore on the axles so that they wobbled and permitted the cars to get off the tracks.

All the ironwork of the cars was made by the Atlas Car Manufacturing Company, and the cars were built on the tunnel works.

The rails in the tunnels were of Carnegie steel section, 25 lb. per yd.; 56-lb. rails were used in the yards. The gauge was 30 in. in the tunnels and 4 ft. 8½ in. in the yards.

A decking was laid in the tunnel, about 3 ft. 9 in. from the under side of the iron, consisting of cross-byats, 12 by 6 in. and 17 ft. long, spaced 5 ft. apart, and longitudinal planking 5 ft. long and 2 in. thick. A bridge of built-up rails was used to conduct the center track into the middle pocket of the shield. This could be lifted during shoving.

Haulage Gear.—For the haulage of tunnel cars on the Manhattan side, four haulage gears, Fig. 2, Plate CIV, were built by the Exeter Machine Works. The requirements were that each haulage gear should be able to handle a wire rope 5 000 ft. long, that is, with a haulage of 2 500 ft., and be able to take 12 loaded tunnel cars, each of 3 700 lb., up an incline of 1½%, and at the same time convey empty cars, or flat cars, loaded with cast-iron segments down the incline.

The attachment between the car and the rope was made with portable grips which were hitched to the bumper of the car, and tripped automatically when the cars reached the air-locks, or the elevators. The rope, at first, was ½ in. in diameter, but was ultimately increased to ⅝ in.

The electrically-driven haulage engine was originally to be stationed in atmospheric air, and the rope was to be depressed under the floor in front of the air-locks, pass through a stuffing-box in the air-tight bulkhead below each air-lock and rise to the floor level again inside under pressure. The motor was a 30-h.p. Crocker Wheeler motor, running on a voltage of 220, of slow-speed type, controlled by a starting rheostat.

The motor, through a countershaft, drove a drum 42 in. in diameter, with six machine-cut grooves for a ½-in. steel cable. In line with this drum was an idler drum having four grooves which were in four independent, sliding, cast-iron rings; any stretching of the rope was accommodated by the slipping of these rings, so that there was no grinding of the rope on the cast-iron surface due to unequal speeds of different parts of the rope.

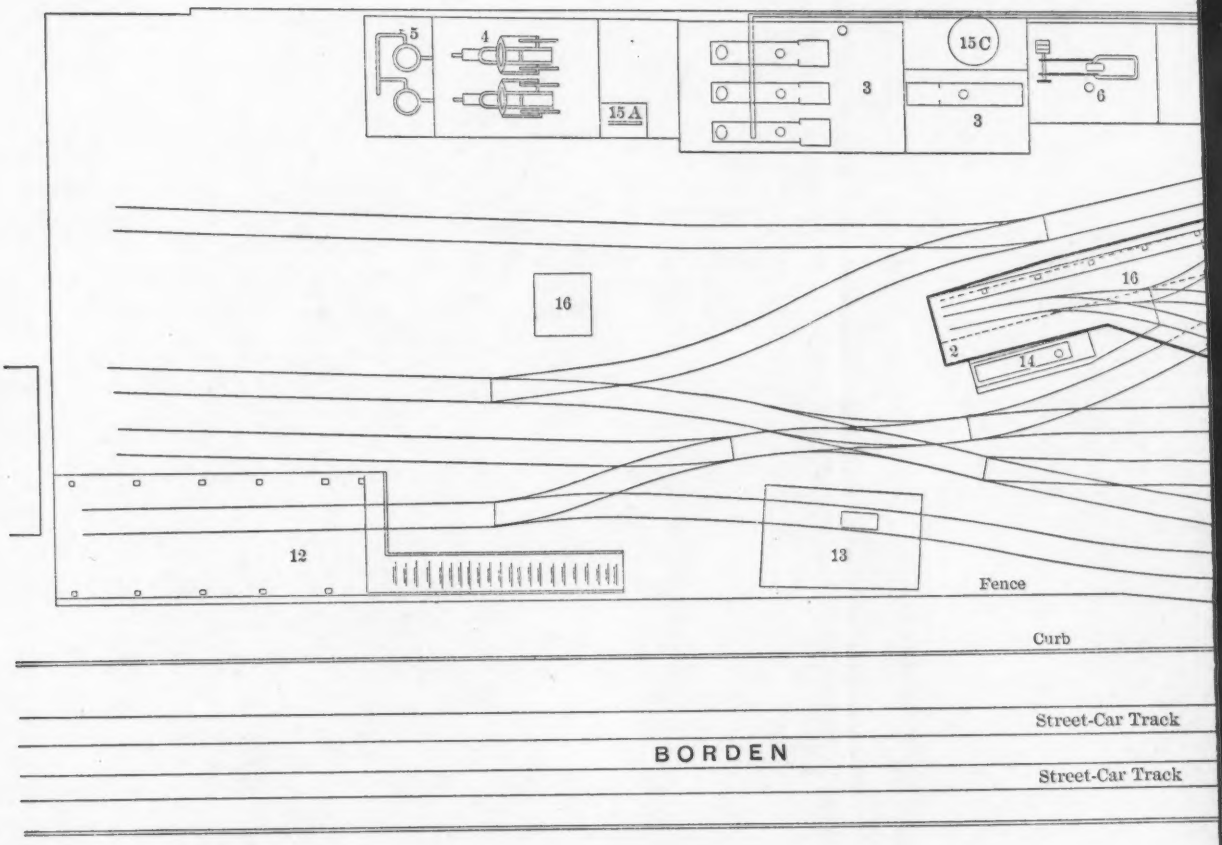
The rope led over a 42-in. grooved sheave which was supported on a traveling tension carriage running on a track in the roof of the tunnel. At the end of the track a weighted tension rope passed over a sheave and kept the haulage rope at any degree of tightness desired.

Trouble was experienced with the friction caused by the rope dipping under the floors so often, and it was not found practical to operate the haulage rope in different air compartments from one engine. In the final arrangement, Plate CV, the engine was mounted in the compressed-air chamber with one part of the rope leading over the tension carriage and back to a sheave over the right-hand muck lock, Fig. 1, Plate CVI, and from there down the tunnel to a sheave under the right-hand forward end of the traveling stage behind the shield. It there passed across the tunnel to a sheave on the left-hand side of the stage and back along the tunnel to a sheave above the left-hand air-lock, and from there was guided to the haulage gear.

The rope was allowed to slide along the floor, supported on rollers to lessen the friction.

Portable grips of many types were tried, but without success, and finally a light ¼-in. chain, with a large square hook on one end which

1. Surface plan of Shaft.
2. Plan of Gantry.
3. Boiler Rooms.
4. Compressors.
5. Air Receivers.
6. Machine Shop.
7. Carpenter Shop.
8. Oil House.
9. Blacksmiths Shop.
10. Cement Shed and Electricians quarters.
11. Powder Magazine.
12. Mule Stable.
13. Engine Shed.
14. Boiler for heating Offices.
15. Contractors Office.
15 A Switch-Board.
15 B Pipe Racks.
15 C Water Tank.
16 Bolt House.
17. Stone-Crushing Plant.
18. Gantry Concrete Mixer.
19. Elevator Shafts.
20. Open shafts for lowering materials.
21. Tipplers.
22. Gantry Derricks.
23. Hydraulic pressure Plant.
24. High-Pressure Air Line from Front St.
24. Low " " " " " "
24. High " Hydraulic Line from Front St.
24. Low " " " " " "
25. Steam Line to open cut
26. Main siding from L.I.R.R.
27. Spur.
28. Open-cut derrick.
29. Hospital Lock.
30. Quarters for Compressed Air Workers.

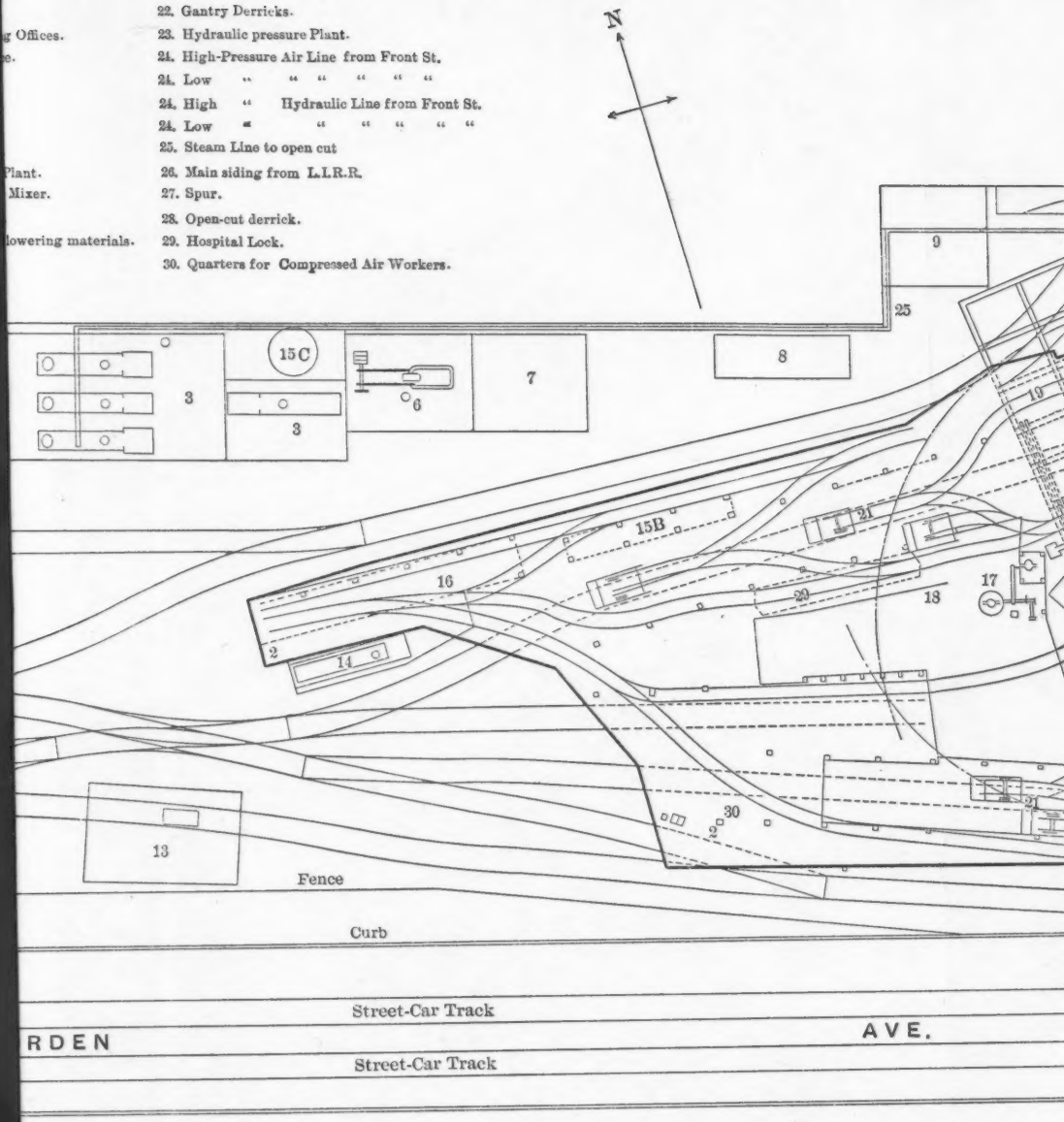


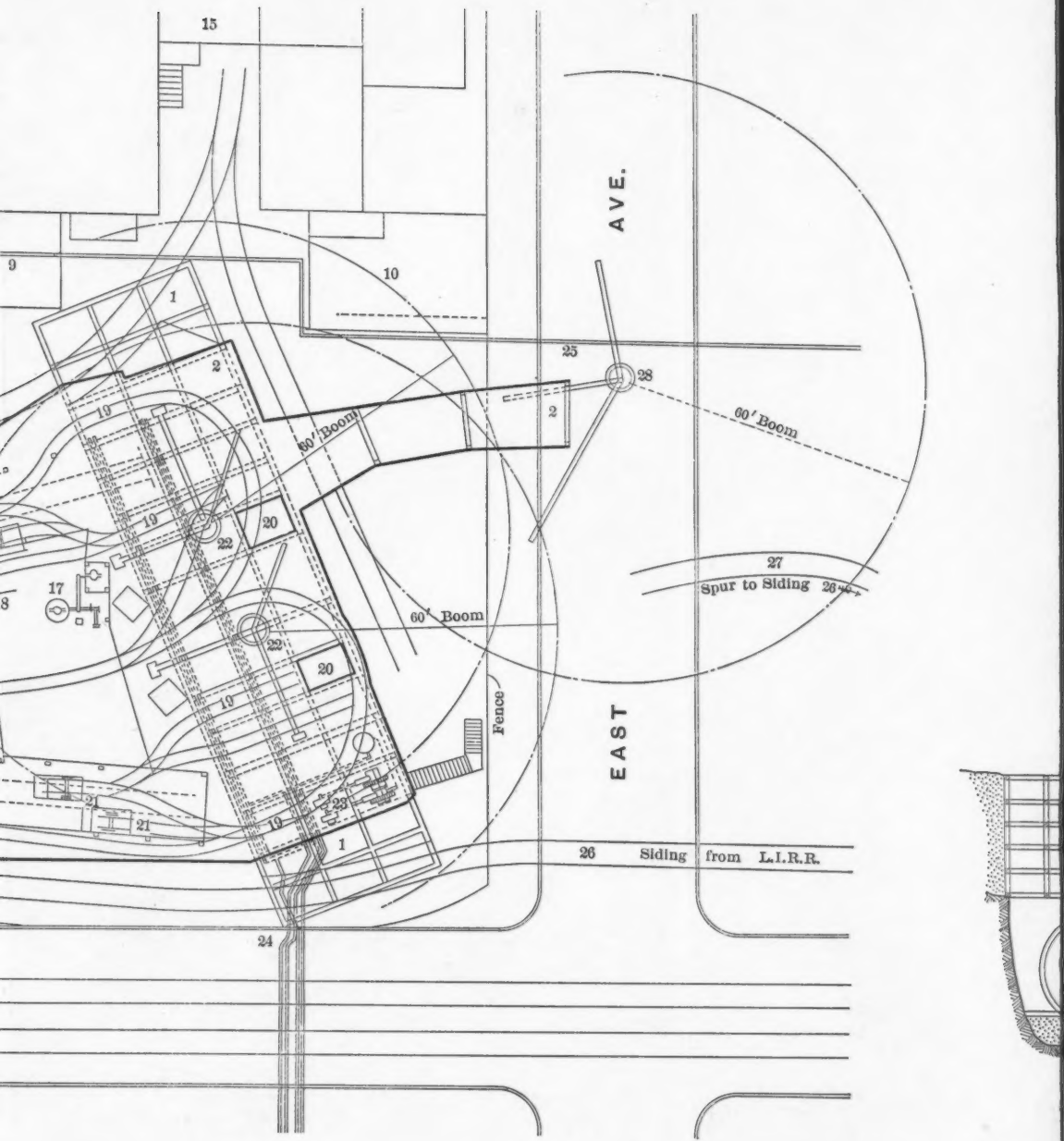
g Offices.
e.

Plant.
Mixer.

lowering materials.

22. Gantry Derricks.
23. Hydraulic pressure Plant.
24. High-Pressure Air Line from Front St.
24. Low " " " " " " "
24. High " Hydraulic Line from Front St.
24. Low " " " " " "
25. Steam Line to open cut
26. Main siding from L.I.R.R.
27. Spur.
28. Open-cut derrick.
29. Hospital Lock.
30. Quarters for Compressed Air Workers.

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17
8
60' Boom
AVE.
25
28
2
60' Boom
27
Spur to Siding 26
60' Boom
Fence
EAST
21
23
19
1
26 Siding from L.I.R.R.
24

15
9
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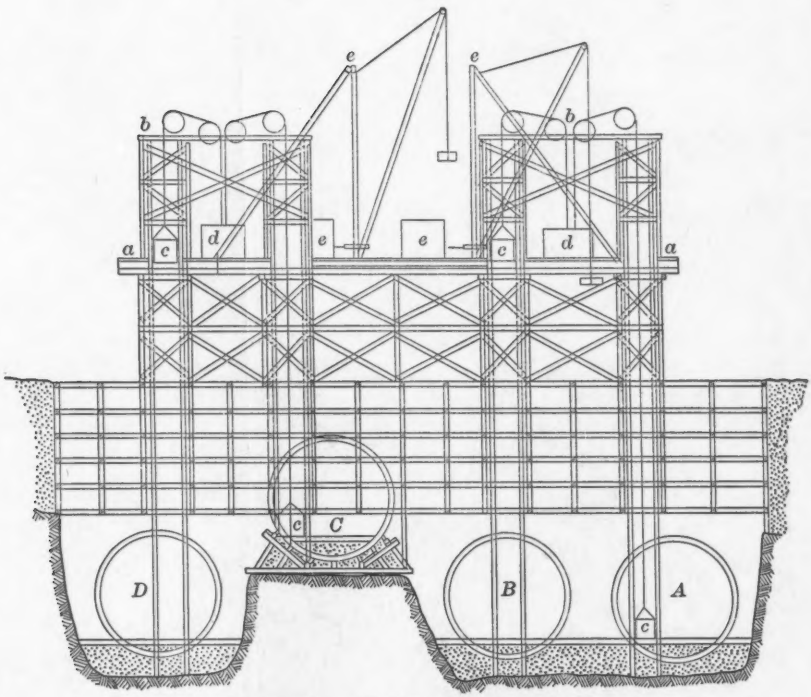
19
19
22
20
17
60' Boom
60' Boom
60' Boom
25
28
2
27
Spur to Siding
22
20
19
Fence
EAST
AVE.
21
21
19
23
1
26
Siding from L.I.R.R.
24

CONTRACTOR'S PLANT
AT EAST AVE.
LONG ISLAND CITY.

A, *B*, *C*, *D* indicate position of Tunnels at West side of shaft.

a Gantry Floor.
b Elevator Frames and gear.
c Cages.
d Elevator Hoist Houses.
e Gantry Derricks and Winches.



SECTIONAL ELEVATION THROUGH EAST AVE. SHAFT.
LOOKING WEST.

hooked over the end of a car while the chain was caught around the haulage rope with a half hitch, proved to be the simplest and most effective method, but it entailed the stopping of the haulage gear for the attachment and detachment of each train of cars.

Inside each bulkhead a small Lidgerwood 7½-h.p. electric or air-hoist was fixed as a stand-by, with a tail-rope which could be drawn down and hitched to the cars if the haulage gear were out of commission. For short distances of 500 ft. between bulkheads a slightly larger double-drum winch, of 10 h.p., either air or electric, Plate CVII, was installed; and, to give a better grip and prevent cutting the drums, cast-steel liners, having a fillet radius of some 6 in., were fixed on the driving drum for the continuous wire rope, which had about four turns on the cast-steel liner, and was led in the same way as the rope from the main haulage engine.

At Long Island City, for the long haul, the Lidgerwood electric hoisting engines, which had been used at East Avenue for the elevators in the shaft and were by that time at liberty, were fitted up as haulage engines, along with two spare ones, by putting a 6-in. fillet around one end of the drum made in sections; the rope, having three turns around this, was led in the same manner as before.

The speed of these hoists was reduced to give a haulage speed of about 3 miles per hour, and the Exeter Machine Company's haulage gears were speeded up from 1 mile to 3 miles an hour.

The haulage in the Manhattan yard was originally done by mules, but the difficulty of getting them about in winter, with heavy loads, and snow and ice on the tracks, made it necessary to install mechanical haulage.

It was determined to try electric haulage, and one of the under frames of the 4-yd. cars was mounted with a pair of axles having steel wheels; then a 15-h.p. electric motor, 250 volts, was fixed on the carriage, gearing to a countershaft on which were two sprocket wheels. A sprocket wheel was also fixed on each axle, and these were driven by chain from the countershaft. A sufficient quantity of cast-iron turnings was put in the bed of the car to give the required tractive effort, and two overhead trolley wires were fitted, the pole having two trolleys insulated from each other.

This method was adopted in preference to bonding the rails, on account of the impracticability of grounding the rails when they

formed part of the lighting system in the tunnels. Two of these cars were made and were very successful.

On the Long Island side, on the surface, where there was ample room, steam locomotives were used, and also at East Avenue. These were made by the Baldwin Locomotive Works, and were of 4 ft. 8½-in. gauge and 5-ft. wheel base, with two 10-in. cylinders having a 16-in. stroke, and four driving wheels, 32 in. in diameter. Steam at a pressure of 160 lb. was generated from 350 sq. ft. of heating surface with 9 sq. ft. of grate area. The total weight in working order was 42 500 lb. The fire-box was of copper and the tubes were of brass. For hauling the concrete in the tunnels, electric traction was adopted.

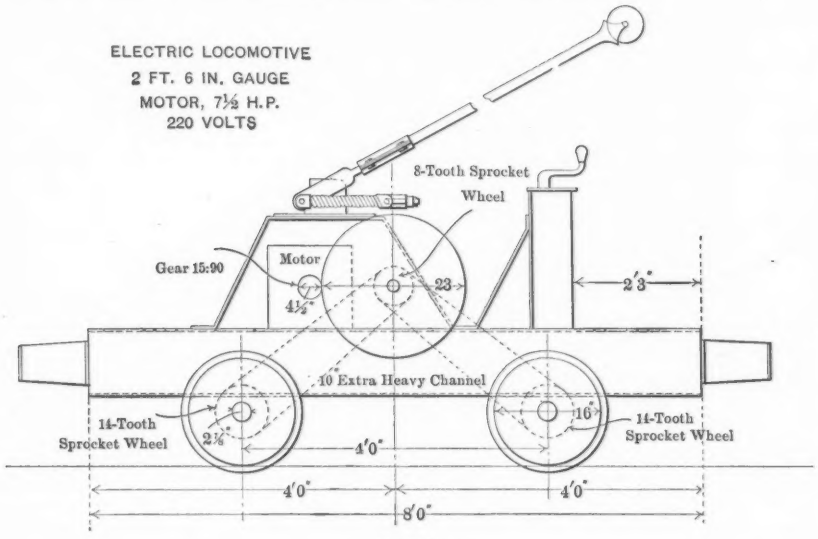
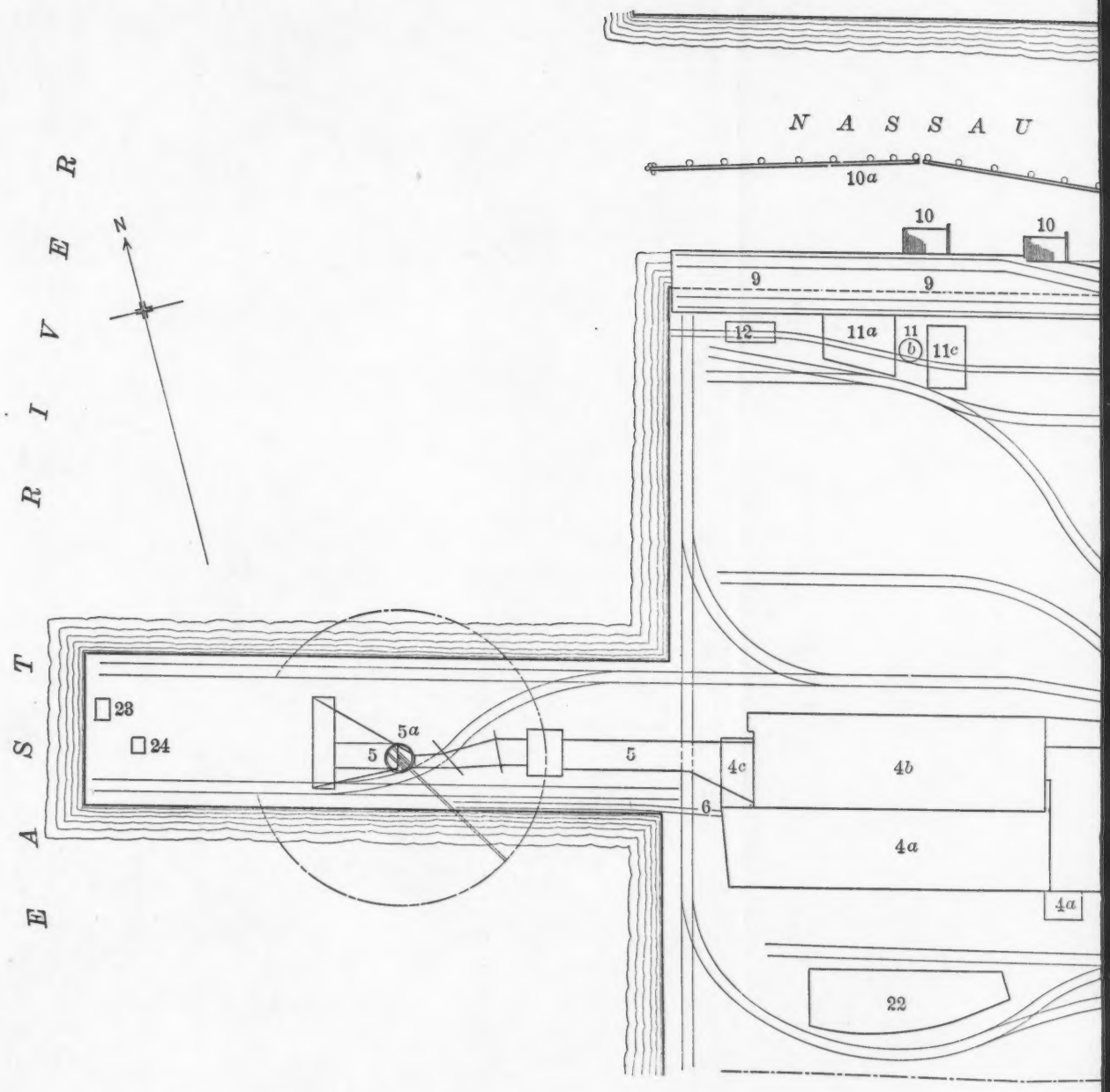


FIG. 18.

Small trolley cars, Fig. 18, 30-in. gauge and 4-ft. wheel base, were built in the tunnel workshop, and a 7½-h.p. motor was mounted, as already described; but, in this case, the rails were bonded, and only a single trolley wire, No. 2, B. & S., of hard-drawn copper was used, the current being supplied from an independent circuit operated by a motor generator of 30 h.p. Each trolley car weighed approximately 3 300 lb. In the writer's opinion, electric traction is more efficient than any other for tunnel work.

Safety Screens and Flying Gangway.—Although the safety screen behind the shield fulfilled the requirements of the specifications to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12
11a
11
b
11c
E A S T R I V E R
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5a
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5
4c
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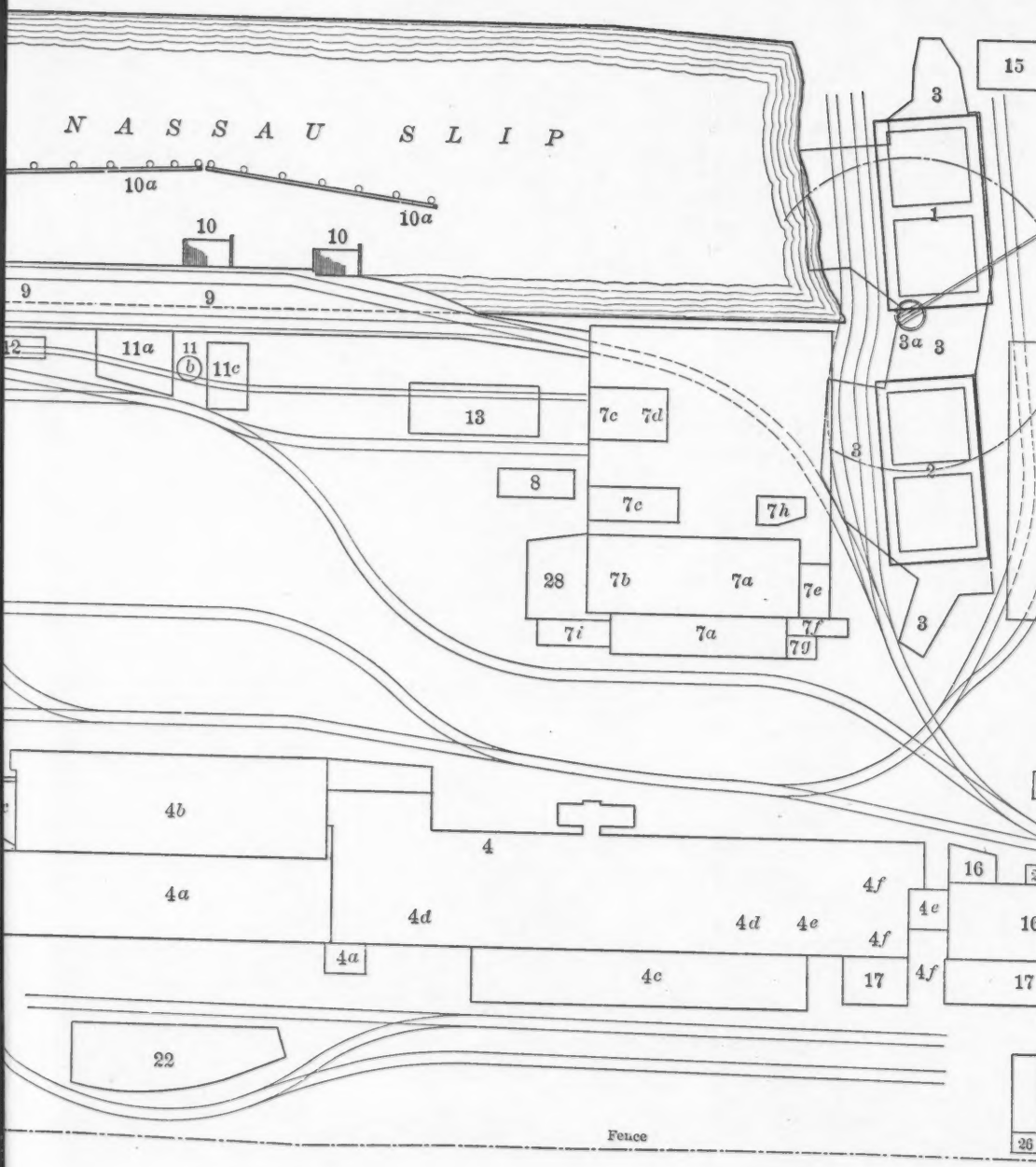
6
4a
4a
22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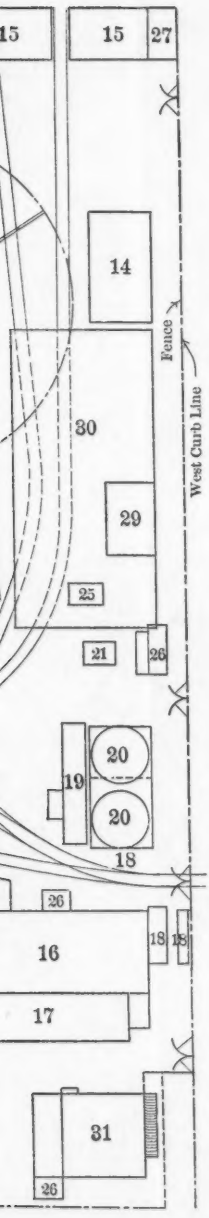
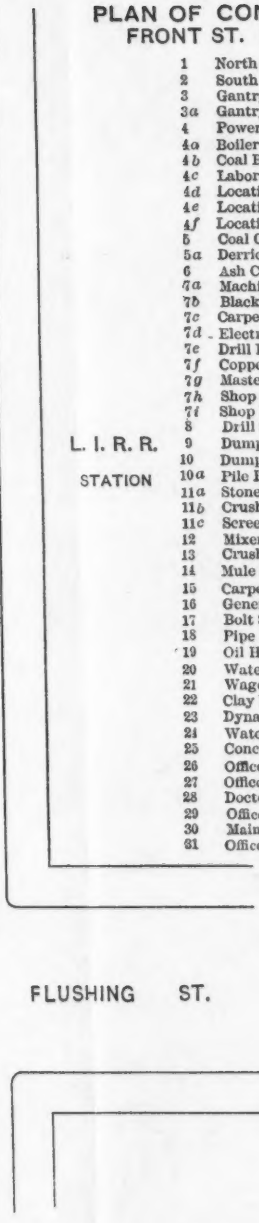
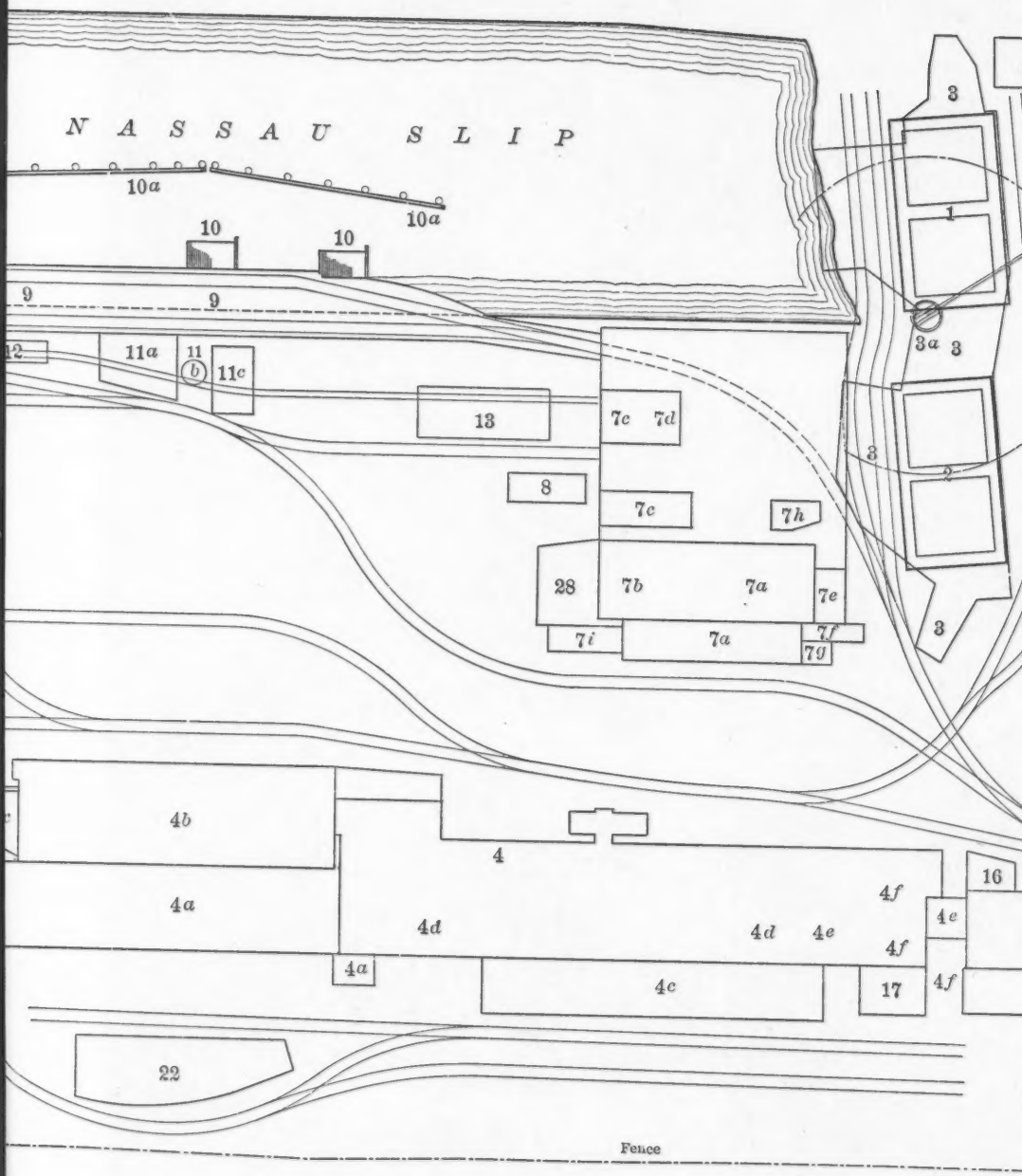




PLAN OF CONTRACTORS PLANT
FRONT ST. LONG ISLAND CITY

1	North Shaft
2	South "
3	Gantry
3a	Gantry Derrick
4	Power House
4a	Boiler Plant
4b	Coal Bunker
4c	Laboratory for Fuel Analysis
4d	Location of Compressed Air Plant
4e	Location of Hydraulic Plant
4f	Location of Electric Plant
5	Coal Conveyor
5a	Derrick
6	Ash Conveyor
7a	Machine Shop
7b	Blacksmiths Shop
7c	Carpenters "
7d	Electricians "
7e	Drill Repair Room
7f	Coppersmiths Room
7g	Master Mechanics Office
7h	Shop Engine
7i	Shop Boiler
8	Drill Sharpeners
9	Dumping Gantry
10	Dumping Screens
10a	Pile Protection
11a	Stone Bin
11b	Crusher
11c	Screen Tower
12	Mixer Plant
13	Crushed Stone Bin
14	Mule Stable and Cement Store
15	Carpenter Shops
16	General Store Rooms and Living Apartments of Master Mechanic
17	Bolt Store Room and Machine Store Room
18	Pipe Racks
19	Oil House
20	Water Tanks
21	Wagon Scales
22	Clay Bin
23	Dynamite Store House
24	Watchmans House
25	Concrete Vault
26	Office Heaters
27	Office of Concrete Sup't and Yard Foreman
28	Doctors Office and Hospital
29	Offices of Shield Superintendents and Time Keepers
30	Main Offices of Contractors
31	Offices of P.T. & T.R.R. Co. Engineers

FLUSHING ST.


NASSAU SLIP
10a
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7g
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4b
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4d
4d 4e
4f
4e
4f
16
4a
4c
17
4f
22
Fence

PLATE CXII.
PAPERS, AM. SOC. C. E.
NOVEMBER, 1909.
JAPP ON
CONTRACTORS' PLANT FOR EAST RIVER TUNNELS.

PLAN OF CONTRACTORS PLANT
FRONT ST. LONG ISLAND CITY

1	North Shaft
2	South "
3	Gantry
3a	Gantry Derrick
4	Power House
4a	Boiler Plant
4b	Coal Bunker
4c	Laboratory for Fuel Analysis
4d	Location of Compressed Air Plant
4e	Location of Hydraulic Plant
4f	Location of Electric Plant
5	Coal Conveyor
5a	Derrick
6	Ash Conveyor
7a	Machine Shop
7b	Blacksmiths Shop
7c	Carpenters "
7d	Electricians "
7e	Drill Repair Room
7f	Coppersmiths Room
7g	Master Mechanics Office
7h	Shop Engine
7i	Shop Boiler
8	Drill Sharpeners
9	Dumping Gantry
10	Dumping Screens
10a	Pile Protection
11a	Stone Bin
11b	Crusher
11c	Screen Tower
12	Mixer Plant
13	Crushed Stone Bin
14	Mule Stable and Cement Store
15	Carpenter Shops
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17	Bolt Store Room and Machine Store Room
18	Pipe Racks
19	Oil House
20	Water Tanks
21	Wagon Scales
22	Clay Bin
23	Dynamite Store House
24	Watchmans House
25	Concrete Vault
26	Office Heaters
27	Office of Concrete Sup't and Yard Foreman
28	Doctors Office and Hospital
29	Offices of Shield Superintendents and Time Keepers
30	Main Offices of Contractors
31	Offices of P.T. & T.R.R. Co. Engineers

some extent, additional safety screens of structural steel were erected in the tunnels behind the traveling stage, and a spare one was built in each tunnel after the shield had moved about 300 ft., leaving one screen to be shifted ahead for the next 300 ft.

The original design of screens was fitted with a light air-lock large enough to take the whole gang of workmen, the forward door of which was shrouded by a curtain that came half way down the air-lock. The doors were hinged to open backward from the shields, because, if the tunnel was flooded for any depth above the bottom of the safety curtain, the pressure of the air between the screen and the bulkhead must necessarily be higher than the pressure between the screen and the shield, or the water would flood the space behind the screen. The curtain was made of $\frac{5}{16}$-in. plates stiffened on the rear by an I-beam, 12 in. deep and 35 lb. in weight, along the bottom edge, with additional stiffening Z-bars horizontally and vertically.

Adjustable segment plates stiffened by a heavy angle bar were drawn up into the circumferential caulking recess of the tunnel lining by eye-bolts suspended from the tunnel bolts, and at the same time the screen was held up by similar eye-bolts passing through heavy angle brackets on the screen. Flanged openings pierced the screen, for the passage of pipes, but it was found easier to depress the pipes under the screen than to cut them to dead lengths to fit the position of the screen.

Although the drawing shows the air-tight joint as having been made by a rubber gasket, the best results were obtained with lead caulking.

The latest type of screen, shown by Fig. 2, Plate CVI, and on Plate CVIII, instead of being fitted with an air-lock, had a square opening, 2 ft. 3 in. by 3 ft. 3 in., the sill of the door being 5 ft. 2 in. above the bottom of the screen, which was set 1 ft. 2 in. above the axis of the tunnel. From the flying gangway a light platform led to this door on either side, the object being to dispense with diving under the curtain if the water rose above the bottom of the screen.

A flying gangway, Fig. 2, Plate CVI, was provided throughout the length of the tunnel, supported by timber hangers from the tunnel lining. It connected with the traveling stage behind the shield by means of a runway, and dipped under the safety screens and connected with the staging at the air-tight bulkheads beside the man-lock. This gave the men a dry means of escape while the water was

rising. No stairway communicated between the floor of the tunnels and this gangway, except near the air-locks, as in emergency it was easily reached by climbing up the flanges of the segments.

Air-locks and Bulkheads.—The concrete bulkheads, Fig. 1, Plate CVI, and Plate CIX, made strong enough to withstand an internal pressure of 50 lb. per sq. in., were built of concrete 10 ft. 6 in. thick. In this were placed two bottom muck locks, one emergency lock, one timber and rail lock, and one engineer's lock for carrying through the lines for surveying the tunnels. In addition, it was pierced by the various pipes and electric cables required for the work in duplicate.

The material air-locks were 20 ft. long and had a mean diameter of 7 ft.; the diameter of the emergency locks was the same, but they were 24 ft. long, as shown on Plate CIX.

It was found during the progress of the work that the 24-ft. lock was bigger than was necessary for the number of men at work, and in the second series of bulkheads the 24-ft. lock was put in place of the right-hand or ingoing material lock. By this change it was possible to take long lengths of pipes, timber, and rails straight through on the cars without the expense of unloading them, passing them through the timber lock and reloading them inside the air-lock.

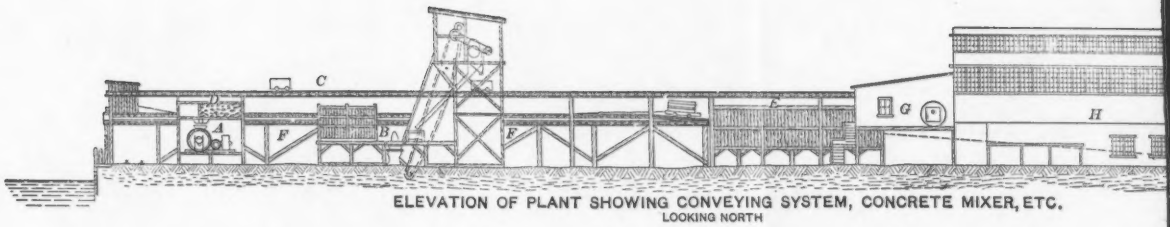
In the bulkheads, where the locks were already fixed, the incoming material locks were lengthened for the same purpose. The locks were made to stand an internal pressure of 50 lb., and that portion of them which extended into the tunnel under pressure from the outside was stiffened to take the same pressure on the outside.

In order to give them a better grip of the concrete bulkhead, and at the same time to take up the external pressure due to the bulkhead acting as a concealed arch, angle rings were riveted outside the locks. Doors giving a clear opening, 5 ft. high and 4 ft. wide, were hinged to swing inward toward the tunnel at each end of the air-locks. They were of the double-dished variety, of the Blackwall tunnel type.

In the emergency lock, brackets were bolted to the shell plates, on which were fixed wooden seats for the men. Bull's-eyes, 6 in. in diameter and 1¼ in. thick, were fitted in the ends of each lock. All three air-locks were fitted with 1½-in. man-valves for the inlet and outlet of air. In addition, the upper lock had a 1½-in. emergency cock which could be opened from the inside of the tunnel to bring back the lock; an additional 1½-in. exhaust valve was also fitted.

LONG ISLAND PLANT

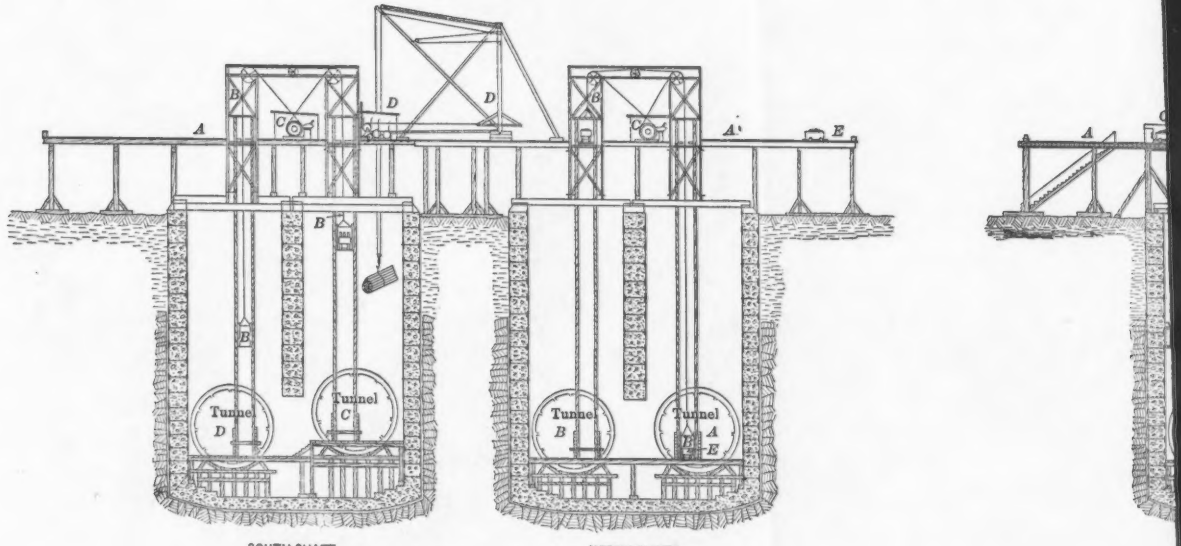
FRONT STREET - LONG ISLAND CITY



ELEVATION OF PLANT SHOWING CONVEYING SYSTEM, CONCRETE MIXER, ETC.

LOOKING NORTH

A-Yard Concrete Mixer
B-Stone Crusher and Screen
C-Runway from Crusher to Bins
D-Mixer Bins
E-Crushed-Stone Bin
F-Dumping Board
G-Doctor's Office
H-Shops and Mens Quarters
I-Switch-Board House
J-Spoil Tippler
K-Spoil Car
L-Gondola Car
M-Elevator Frame and Cage
N-Hoisting-Engine House
O-Gantry Derrick
P-Contractor's Main Office
Q-Yard Material Flat Car
R-Tunnel Spoil Car
S-Engineer's Instrument Tower
T-Shaft Bins for Crushed Stone and Sand
U-Shaft Elevator for Concrete Plant
V-Shaft Mixer used for Concrete Lining
W-Tunnel Concrete Cars



SOUTH SHAFT

NORTH SHAFT

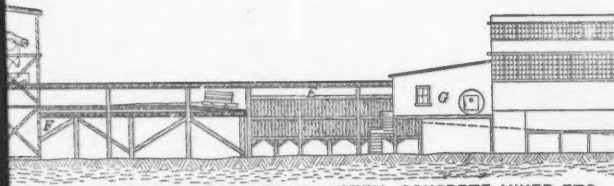
SECTIONAL ELEVATION THROUGH SHAFTS AND GANTRY

LOOKING WEST

A-Gantry Floor
B-Elevator Framing and Cages
C-Hoisting Engine Houses
D-Gantry Derrick and Hoisting Engine
E-Tunnel Spoil Car

LONG ISLAND PLANT

FRONT STREET - LONG ISLAND CITY

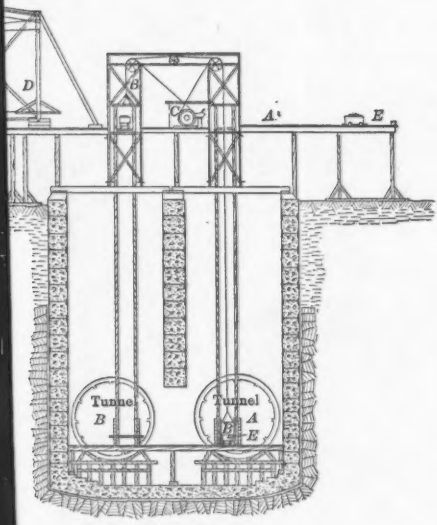


ON OF PLANT SHOWING CONVEYING SYSTEM, CONCRETE MIXER, ETC.

LOOKING NORTH

poil Tippler
poil Car
ondola Car
levator Frame and Cage
oisting-Engine House
antry Derrick
ontractor's Main Office
ard Material Flat Car
unnel Spoil Car

S-Engineer's Instrument Tower
T-Shaft Bins for Crushed Stone and Sand
U-Shaft Elevator for Concrete Plant
V-Shaft Mixer used for Concrete Lining
W-Tunnel Concrete Cars



NORTH SHAFT

UGH SHAFTS AND GANTRY

WEST

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and Hoisting Engine
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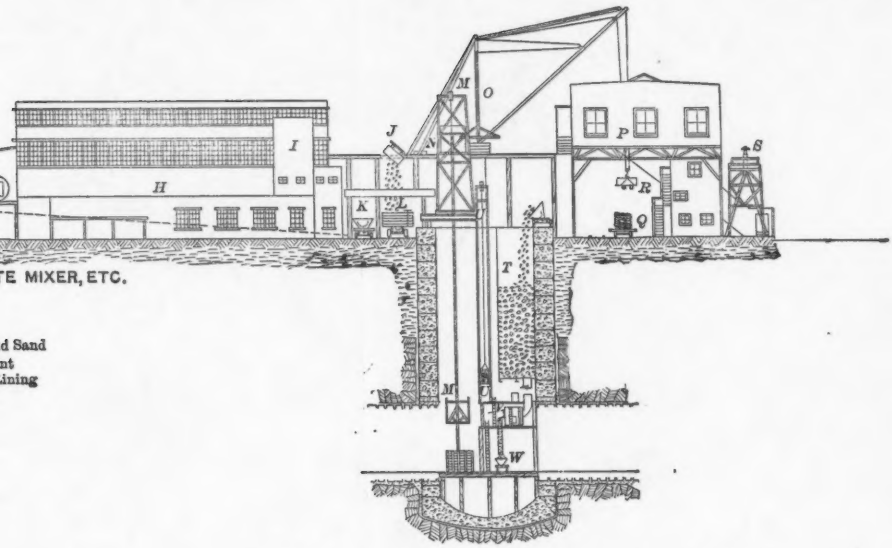
PLATE CXIII.
PAPERS, AM. SOC. C. E.
NOVEMBER, 1909.
JAPP ON
CONTRACTORS' PLANT FOR EAST RIVER TUNNELS.



TE MIXER, ETC.

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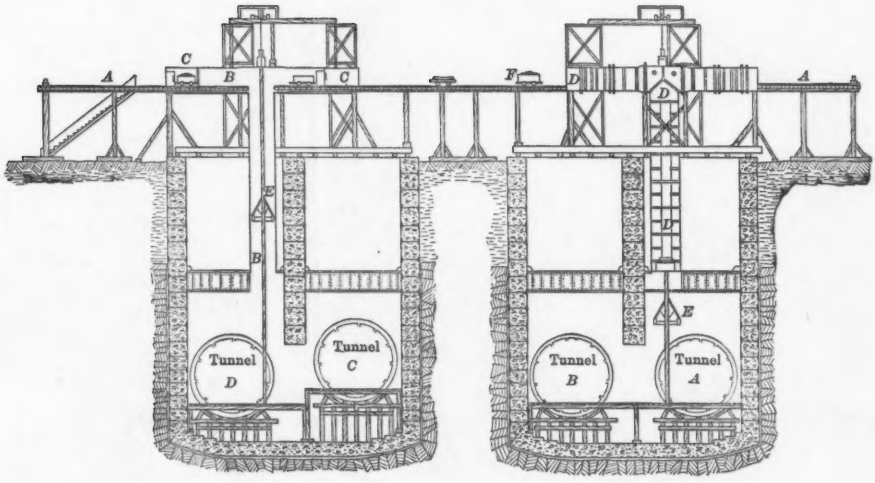


SOUTH SHAFT NORTH SHAFT

SECTIONAL ELEVATION THROUGH SHAFTS AND GANTRY
LOOKING WEST

A-Gantry Floor
B-Sectional Elevation of T-Head Shaft and Locks
C-Locks
D-Outside Elevation of T-Head Shaft and Locks
E-Elevator Cages
F-Tunnel Spoil Car

A clock and a pressure gauge were fixed in each air-lock for guiding the men as to the rate of decompression, and an automatic decompressing valve was also attached. The details of these decompressing valves will be found in a paper by the writer entitled "Caisson Disease and Its Prevention."*

For locking out materials, 3-in. cocks were fitted, but their diameter was ultimately increased to 4 in. In order to safeguard the workmen who used the muck locks for entering or leaving the tunnels, these 4-in. cocks had their counterpart inside the lock, so that the men could close them and operate the lock by means of the man-valves. The track was continued up to the well in which the lock door swung, and on the Manhattan side, this was bridged by two short lengths of independent rail which fitted into a recess on the door sills, and could be very quickly placed in position by the lock tender. On the Long Island side, two short rails were coupled together and hinged so that they could be swung up out of position to allow the door to open. Either method works well.

The pipe lock, which proved ultimately to be unnecessary, and expensive to operate, was 31 ft. long, of steel pipe, 21 in. inside diameter, fitted on the inside with a hinged door opening inward to the tunnel and seating on a rubber gasket against the flange of the pipe. On the outer end there was a box 2 ft. 10 in. square and 27 in. long with an opening 17½ in. square with a door opening inward.

Cross-Passage Locks.—Communication locks between tunnels were fitted at Long Island City to operate with the pressure either way (Plate CX).

Blow-out Lines.—Two 6-in. water blow-out lines, and one 6-in. foul-air blow-out line passed along each tunnel. Branches were fitted at each bulkhead to take out any water collecting there, and at the shield a stuffing-box was bolted to the pipe in which was a telescopic pipe fixed rigidly to the traveling stage. From this a 6-in. flexible pipe dropped to the invert with a strainer on the end of it. This very quickly drew out any water that collected, and in the quicksand section took a great quantity of sand, which was blown along with the water into the sump at the foot of the shafts.

On account of the sand passing through the pipes, they were very quickly cut away, and at the bulkheads this was a serious matter, as

* *Transactions*, Am. Soc. C. E., Vol. LXV, p. 1.

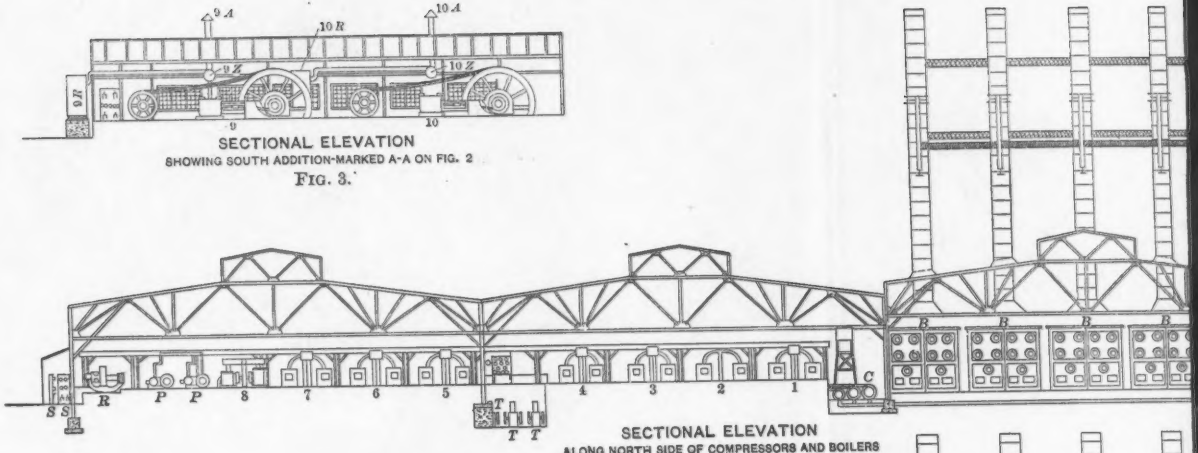
there was no way to replace them. Advantage was taken of two 8-in. spare pipes which passed through the bulkhead near the floor level, and extra heavy 6-in. pipes were passed through these 8-in. pipes and flanged, and the annular space between caulked with lead. It was always possible to renew this extra-heavy internal pipe in case it was cut out by sand.

Many minor accidents occurred to workmen who passed close to the end of the blow-pipes when the strainer was off. Similar accidents took place at the big muck valves of the material air-locks. It was common for a man, with his coat over his arm, waiting for the lock to be sent back, to have it snatched off and sucked through the 4-in. valve into the air-lock.

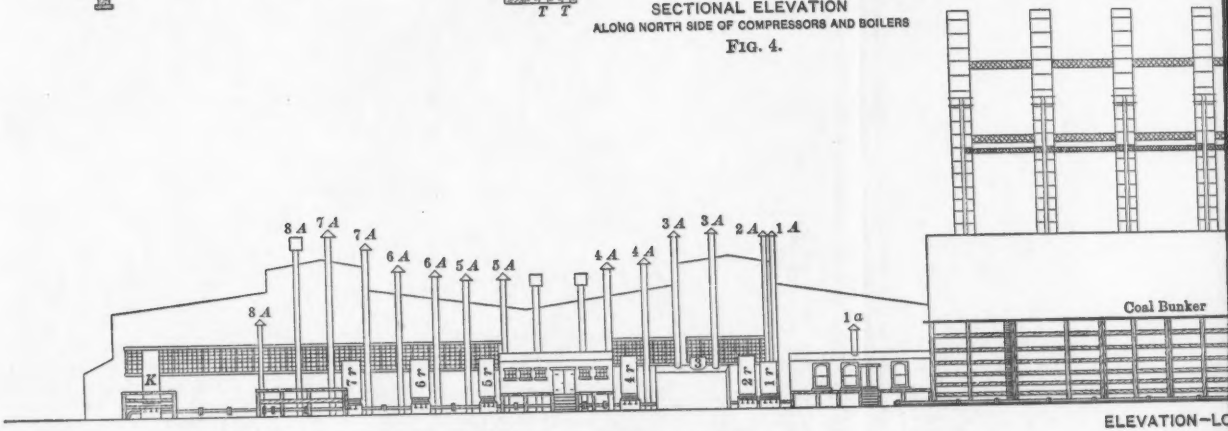
Caisson Air-locks.—For sinking the caissons, Moran-Barr air-locks, leased from the John F. O'Rourke Company, were used. The upper door of this lock is in two parts and slides together, meeting in the center of the opening around a brass bushing on the cable. The sliding of the doors is accomplished by air cylinders, and the lower door is a simple hinged flat door opening inward toward the caisson by hand. These locks worked exceedingly well and rapidly. The maximum number of ½-cu. yd. skips taken out of one caisson fitted with two air-locks, but with only one crane at work, was 256 in 8 hours. When the caisson was just started, the pressure being about 10 lb., it was quite easy to open the upper sliding doors, although the pressure of the caisson was against them. On three occasions, the lock tenders, in their desire to hustle things, opened the upper sliding doors before the bottom door was properly closed, the result being that the skip of muck was violently shot up in the air. As this might have resulted in the loss of all the air in the caisson, with consequent flooding and drowning of the workmen, an interlocking gear was designed, consisting of a connecting link on the lever of the bottom door controlling a pin which prevented the opening of the upper sliding doors until the lower door was fully closed. This effectually prevented such accidents.

The general details of these air-locks have been illustrated so frequently that it is unnecessary to describe them here. They were mounted on an elliptical shaft divided with a partition to give room for ½-cu. yd. buckets to pass up and down freely, and on the other side of the partition a ladderway for the workmen was fixed. Two sets of shafting, with an air-lock on each, passed through the air-tight

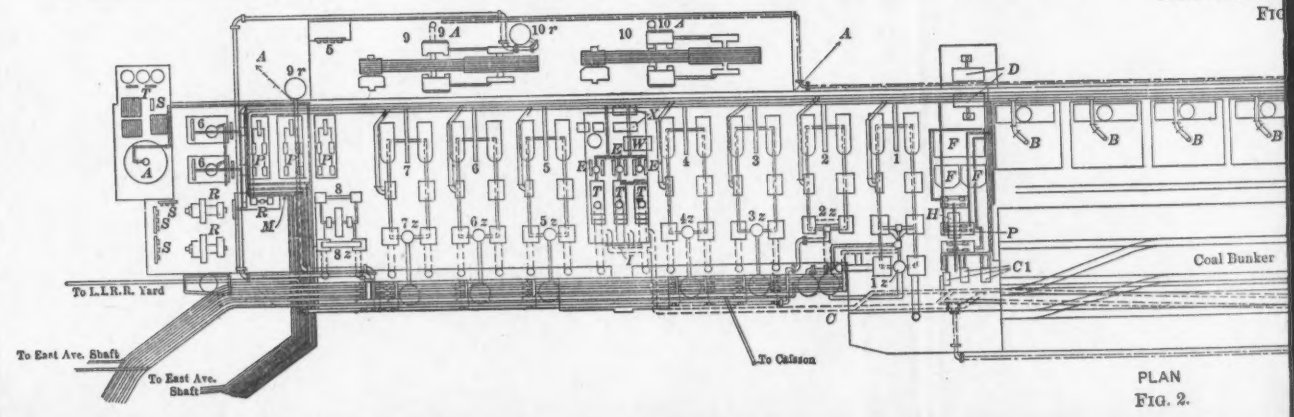
SECTIONAL ELEVATION
SHOWING SOUTH ADDITION-MARKED A-A ON FIG. 2
FIG. 3.



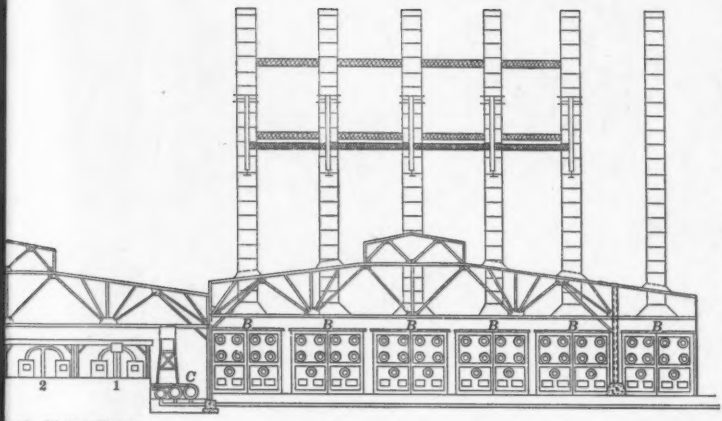
SECTIONAL ELEVATION
ALONG NORTH SIDE OF COMPRESSORS AND BOILERS
FIG. 4.



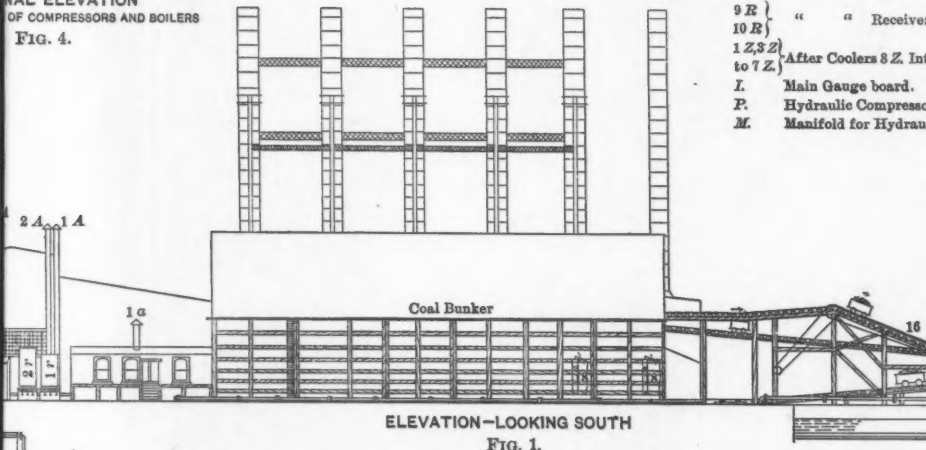
ELEVATION—LO
FIG



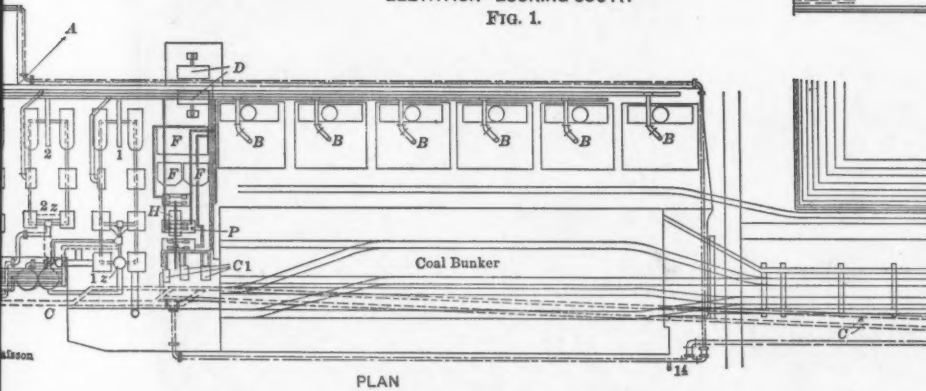
PLAN
FIG. 2.



NAL ELEVATION
OF COMPRESSORS AND BOILERS
FIG. 4.



ELEVATION—LOOKING SOUTH
FIG. 1.



PLAN
FIG. 2.

FIG

1 *A* to 8 *A*.	Fresh Air intakes to C
1 *R*. 3 *R* to 7 *R*.	Low Pressure Air Re
2 *R*.	Receiver for High Pr
K.	Receiver—after cooler
C 2.	Worthington Emerge
C 3.	Electric Turbine
13.	Intakes
14.	City Water Connecti
15.	Coal Hopper and Scal
16.	Coal Elevator.
17.	Dynamite Store-hous
17 *a*.	" "
18.	Coal Chutes for yard

F

1.	Combination High a
2.	High Pressure Comp
3 to 7.	Low "
8.	High "
9.	Low Pressure Motor
10.	" " "
9 *R* 10 *R*	" " Receive
1 *Z*, 3 *Z* to 7 *Z*.	After Coolers 8 *Z*. In
I.	Main Gauge board.
P.	Hydraulic Compress
M.	Manifold for Hydrau

CONTRACTORS POWER PLANT

FRONT ST., LONG ISLAND CITY.

NOTE:-
Numbers and letters always apply to the same object in all figures on this sheet.

FIG. 1.

Fresh Air intakes to Compressors.

Low Pressure Air Receivers.

Receiver for High Pressure Air.
Receiver–after cooler
Worthington Emergency Circulation Pumps.
Electric Turbine " "
Intakes " "
City Water Connection to " "
Coal Hopper and Scales House.
Coal Elevator.
Dynamite Store-house.
" " watchmans shed.
Coal Chutes for yard locomotives.

FIG. 2.

Combination High and Low Pressure Compressor.
High Pressure Compressor (changed to Low Pressure).
Low " "
High " "
Low Pressure Motor driven Compressor.
" " " " "
" " Receiver.

After Coolers 8 *Z*. Intercooler.

Main Gauge board.
Hydraulic Compressors.
Manifold for Hydraulic Power System.

FIG. 2. continued.

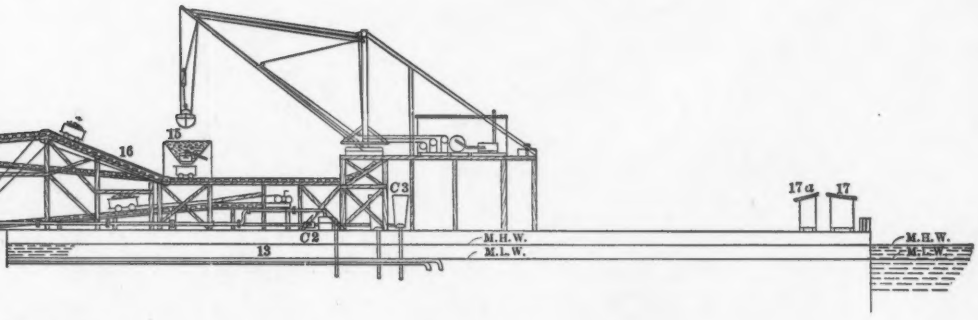
G. Electric Generators.
R. Rotary Converters.
S. Switch Boards.
B. Boilers.
D. Forced Draft Fans.
F. Feedwater Filters.
P. " Pumps.
H. " Heater.
C. Cold water Circulation System (in part).
C 1. " " " Pumps.
C 2. Cold water Emergency Circulation Pumps. Worthington.
C 3. Cold water Emergency Circulation Pumps. Electric Turbine.
T. Condensers.
E. " Air Pumps.
W. Hot Well.
X. " " Pumps.
O. Oil Fillers.

FIG. 3.

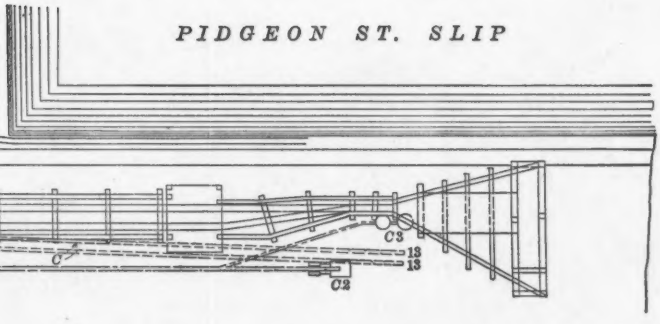
9. Low Pressure Compressors.
10. " " "
9 *A*. Fresh Air Intake.
10 *A*. " " "
9 *R*. Receiver for No. 9 Compressor.
10 *R*. " " No. 10 "
9 *Z*. After cooler.
10 *Z*. " "

FIG. 4.

C 1. Circulation Pumps.
C. " Suction Pipe Line.
B. Boilers



PIDGEON ST. SLIP



floor, which consisted of steel girders at 5-ft. centers, 29 ft. long and 6 ft. deep supporting ½-in. steel buckle-plates with a rise of 6 in.

So many accidents have occurred through fire, with wooden air-tight floors, that the contractors determined to go to the extra expense of the steel buckle-plates.

After the caissons were sunk and sealed, the air-tight floors were removed and the shields were built in position on wooden cradles. When they were built, a full set of tunnel air-locks was lowered beside each shield, and the air-tight floor was rebuilt above the shields, and these air-locks were hoisted up, and suspended underneath the floor, out of the way, as shown by Fig. 19.

After the shields had advanced 500 ft., the tunnel air-locks were lowered and built in position for the first bulkhead. This saved a heavy item for labor over what it would have cost to lower the air-locks in sections through the shaft air-locks, transport them along the tunnel, and build them in position.

When the air-tight floors were rebuilt, it was considered that the O'Rourke air-locks, hoisting ½-cu. yd. skips, would be inadequate for handling the excavation from two shields. It was decided, therefore, that tunnel muck cars should be used, and an elevator was installed in a vertical shaft which opened out into a T-headed air-lock, Fig. 19. The vertical shaft was built up of air-lock material which was needed later for tunnel bulkheads, with the addition of a steel plate about 3 ft. wide, which increased the diameter from 7 to 8 ft.

A special miter section crowned the 8-ft. vertical shaft, and, reducing to a diameter of 7 ft., branched out on either side into two standard horizontal air-locks, each 12 ft. long, giving enough space for one tunnel car at a time.

The cage, in the vertical shaft on timber guides, was designed to pass between two of the air-tight floor girders.

To one side of the vertical shaft, partitioned off from the space occupied by the cage, two vertical steel ladders were fitted to give the workmen access to the tunnels if the cage were out of order.

To give the workmen sufficient head room when the cage reached the top, a well was built underneath the cage track, so that they could stand down. The hoisting rope and the shackle on the over-winding gear of the cage passed up into a dome on top of the miter piece, fitted with a stuffing-box on top. As the shields were working in the

solid rock face at this time, one cage was able to take care of the muck from two shields. The vertical access shaft and T-headed locks were placed so that it was possible to build the regular pit-head gear above the caissons, with everything in readiness so that when the air was removed from between the first bulkhead and the air-tight floor, it required only an off Sunday to change over from the T-headed lock system to a straight hoist in atmospheric air with one cage for each tunnel. The air-tight floor was then removed at leisure, part of it being retained for use later as the bottom floor of the stone and sand bins for concreting operations.

For hoisting the spoil from the tunnels, one cage, Fig. 20, was provided for each tunnel, namely, four at Manhattan, four at Long Island City, and four at East Avenue. As the tunnels were in pairs in each shaft, the cages were operated in pairs off the same winch, so that when one cage was lowered the other one was raised. When the tunnels of a pair were at different levels, as at Long Island City and East Avenue, the difference in height was taken care of by having the drum divided into two sections of different diameters in the ratio of the heights, the greatest ratio being as 72 ft. is to 53 ft. for the *C* and *D* East Avenue cages.

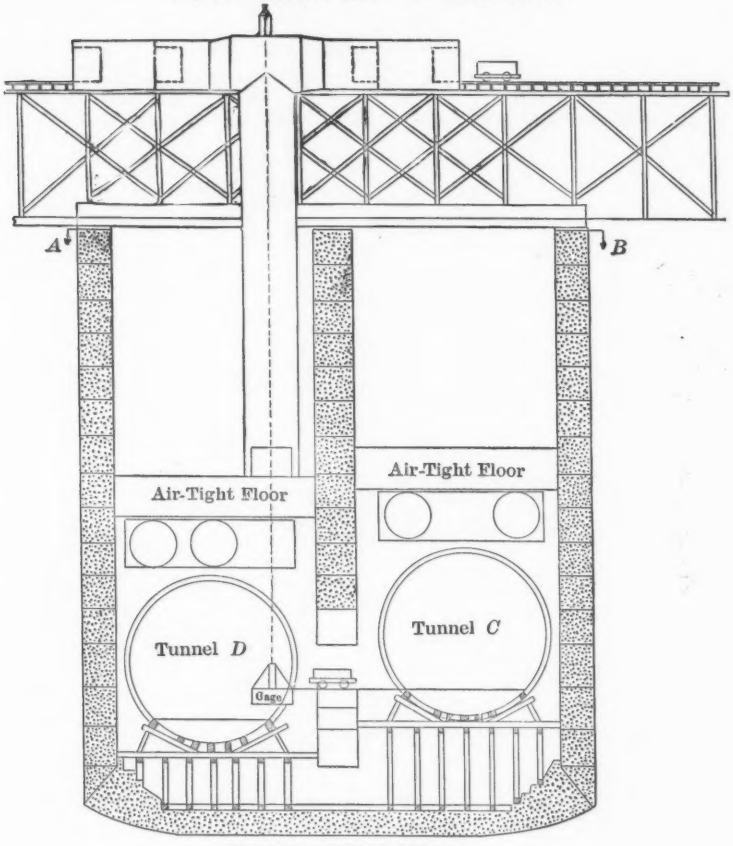
The hoists were capable of lifting an unbalanced load of 2 700 lb. at a speed of 300 ft. per min.; 3 700 lb. at 200 ft.; and 5 000 lb. at 150 ft. The standard drums were 42 in. in diameter, and had a capacity for 140 ft. of wire cable $\frac{7}{8}$ in. in diameter, the drum being grooved to receive it.

The hoist was fitted with one band-brake and a solenoid-brake which went into operation when the electric circuit was broken from any cause. The hoists were built by the Lidgerwood Manufacturing Company, and the motors were supplied by the General Electric Company. They were of the fully-enclosed railroad type, wound for 220 volts, and could stand 100% overload for 1 min. without injurious sparking. They were operated by a controller with a reversing switch.

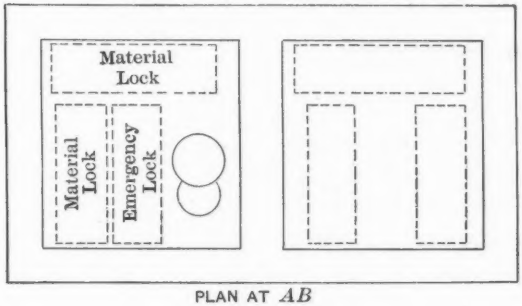
The cages, Fig. 20, were built of light 2-in. angle bars and were guided on 6-in. vertical timbers. If the cage dropped too quickly, either through accident or otherwise, toothed cams gripped the guides automatically and brought the cage to rest. King and Humble's patent safety over-winding catches were fitted overhead.

As an additional precaution, a wire rope was passed from one cage

NO. 2 CAISSON WITH T-HEAD LOCK



SECTIONAL ELEVATION



PLAN AT *AB*

FIG. 19.

to the other over the pit-head sheaves, which were double-grooved; and, midway between the two pit-heads, overhead, it passed four times round a drum fitted with a band-brake controlled by a lever in the hoisting-engine house. In the event of the hoisting rope breaking while the cage was being elevated, the loaded cage on falling pulled up the

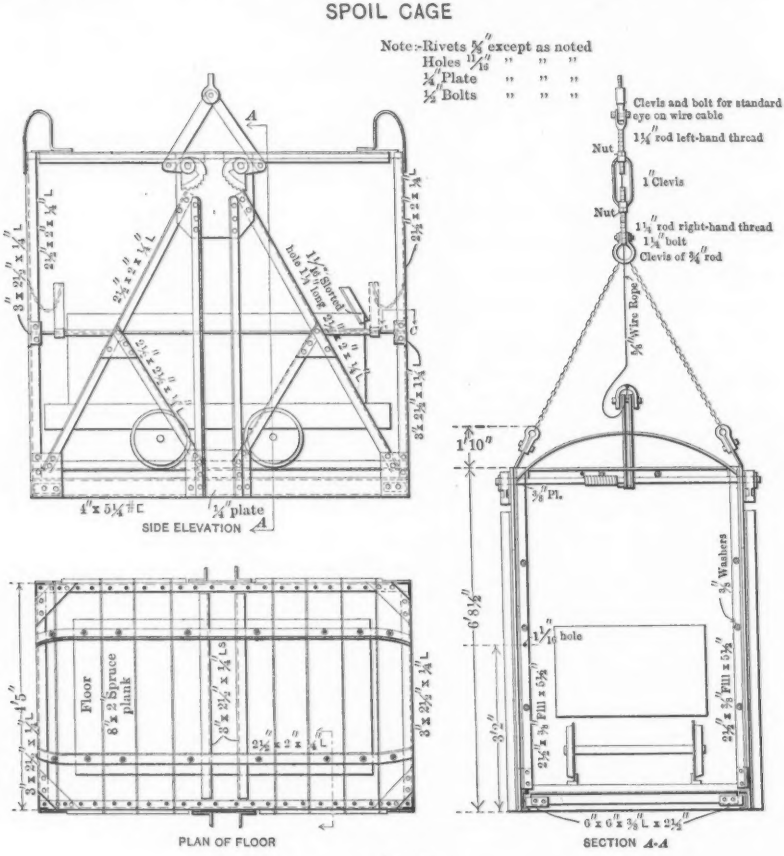


FIG. 20.

empty cage by means of the additional rope. When this occurred, the operator put his foot on the lever controlling the band-brake of the free drum, and brought the cages to rest without accident, this being necessary only if the automatic grips failed to act.

To prevent anything falling down the pit, a light wooden cover rested on the upper floor and slid on the guides. This was lifted clear

(FIG.1)

1 Radley Building
2 Grout Machine House
3 Crushed Stone Bins
4 Switch Board
5 Stone Crusher
6 S.Pearson & Son Locker B'ld'g
7 Switch Board
8 Stables
9 Cement Bag House
10 Hoisting House
11 Chute for Stone
12 Water Tanks
13 S.Pearson ,& Son Office

PLAN OF
SECOND-FLOOR GANTRY

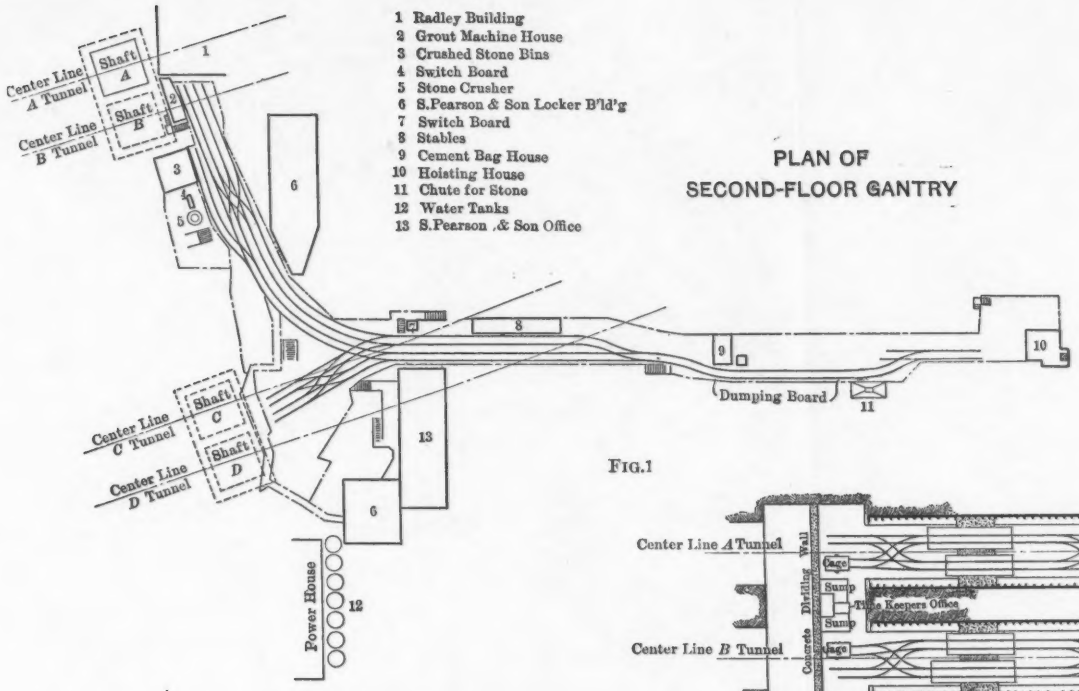


FIG.1

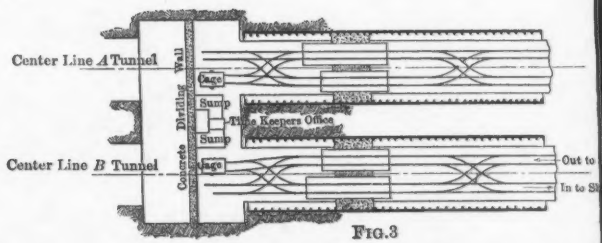


FIG.3

PLAN AT BOTTOM OF *A* AND *B* SHAFT SHOWING TRACK LAYOUT.
CAGES AND MATERIAL LOCKS IN FIRST BULKHEAD

(FIG.2)

1 Radley Building
2 Hoist House
3 Tipplers
4 Hoist House
5 Crushed Stone Bins
6 Boiler
7 Boiler
8 Hoist House
9 Cement House
10 Boiler
11 Hoist House
12 Boiler
13 Telephone Box
14 Operators Tower
15 Tipplers for Crusher
16 Derricks
17 Coal Bins
18 Tower for Ashes
19 Ash Chute

THIRD-FLOOR GANTRY

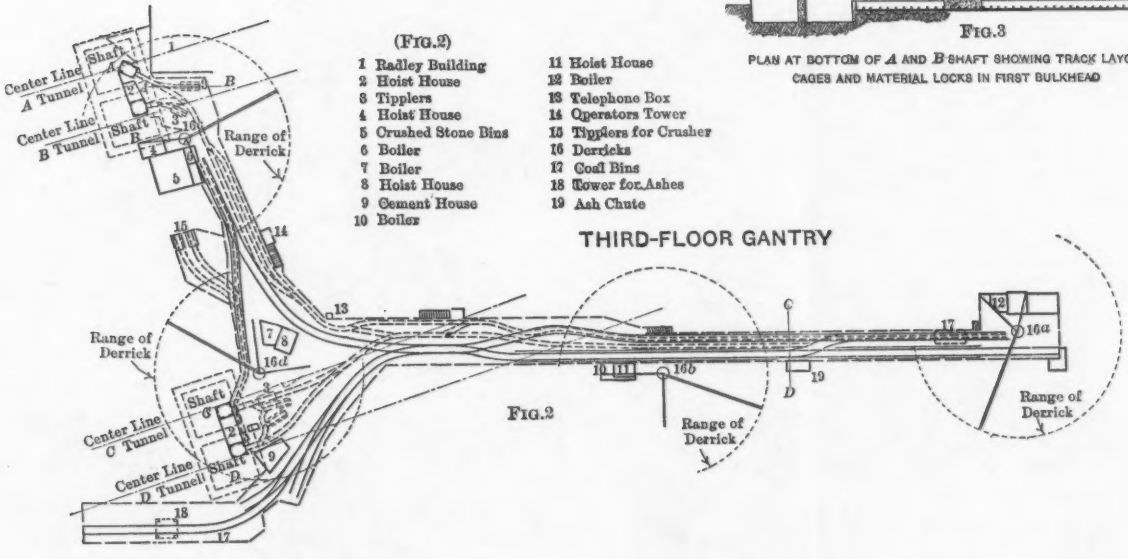


FIG.2

uilding
chine House
Stone Bins
oard
sher
& Son Locker B'ld'g
oard

ag House
House
Stone
nks
.& Son Office

PLAN OF SECOND-FLOOR GANTRY

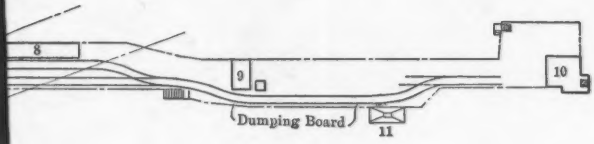


FIG.1

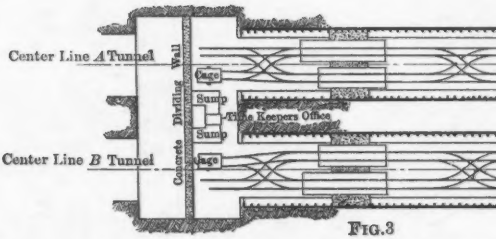


FIG.3

PLAN AT BOTTOM OF *A* AND *B* SHAFT SHOWING TRACK L
CAGES AND MATERIAL LOCKS IN FIRST BULKHEAD

11 Hoist House
12 Boiler
13 Telephone Box
14 Operators Tower
15 Tipplers for Crusher
16 Derricks
17 Coal Bins
18 Tower for Ashes
19 Ash Chute

THIRD-FLOOR GANTRY

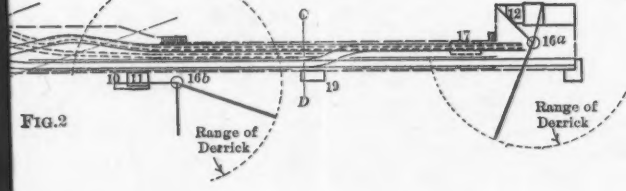


FIG.2

PLATE CXV.
PAPERS, AM. SOC. C. E.
NOVEMBER, 1909.
JAPP ON
CONTRACTORS' PLANT FOR EAST RIVER TUNNELS.

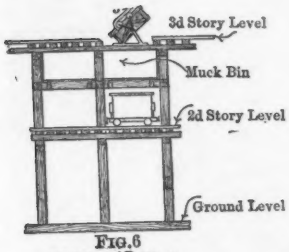


FIG. 6
SECTION *AB* FIG. 2

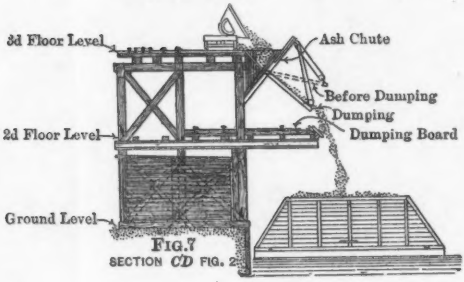


FIG. 7
SECTION *CD* FIG. 2

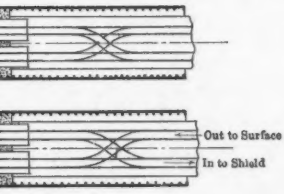
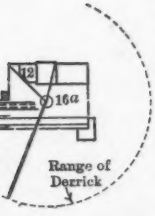


FIG. 3
AFT SHOWING TRACK LAYOUT.
IN FIRST BULKHEAD



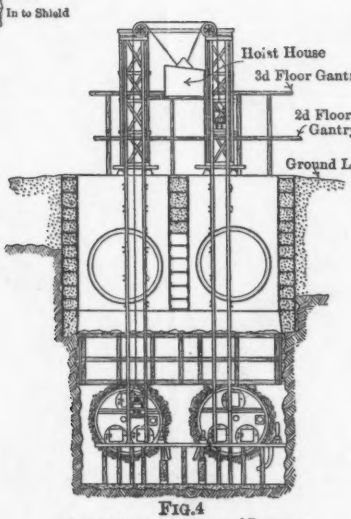


FIG. 4
SECTIONAL ELEVATION OF *AB* SHAFT
LOOKING EAST

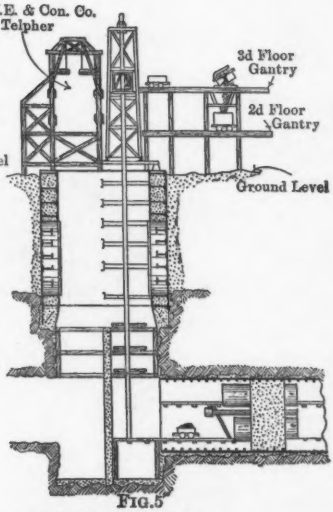


FIG. 5
SECTIONAL ELEVATION OF TUNNEL
B AND SHAFT
LOOKING NORTH

by the rising cage. On the upper level, the cage rested on tumblers, which closed automatically when the cage passed through them. At the bottom, an automatic gate lifted when the cage came down, and fell into position when the cage rose. These were apt to get out of order, and the most usual method for protecting the bottom was with a light piece of chain hung across the opening. The cars were kept in place in the cage by two swing-bars. The loaded muck cars, on reaching the surface, were pulled out by hand and run into a tippler where they upset automatically and dumped into muck bins. The tippler was built very strongly of 56-lb. rail, and was designed so that, with the loaded car, the center of gravity was such that the car and tippler turned upside down, and, when the car emptied, the center of gravity was such that the car came back to its normal position. If the muck was sticky, it was necessary to catch the tippler upside down on its swing.

Pumping Plant.—For the compressed air tunnels, no pumps were necessary in the air chamber, as the air pressure blew the water out through pipes to the sump. It was possible, under special circumstances, by allowing air to leak into the pipe from the chamber, for the water to be delivered right up into the river without the use of pumps, but generally it was more reliable to blow the water from the tunnel into the shafts and pump from there.

At the foot of each shaft, as a stand-by in the event of flooding, one special 6-in. vertical turbine pump was installed. These pumps were capable of delivering 60 000 gal. per hour. Three of them were built by the Worthington Company, and two by R. D. Wood and Company, the motors, of 50-h.p. each, being made by the Northern Electric Company. The motors were enclosed in a water-tight chamber.

For ordinary service work, the most suitable pump for the rough work and large volumes of water proved to be the Cameron No. 12, with 18-in. steam and 12-in. water cylinders, and a 20-in. stroke. These were the main stand-bys on the Manhattan side, while on the Long Island side, Blake duplex pumps, with 14-in. steam and 12-in. water cylinders, and a 10-in. stroke, were installed.

At the East Avenue site of the works, a great number of pumps were necessary in the headings and break-ups, and outside the air-tight bulkheads at the lower end of this section. The No. 9 Cameron pump

was generally adopted, although smaller sizes were used at various points. These pumps take up so little room that they can stand at the side of the headings without interfering with the passage of cars.

As the operation of pumps by air pressure is very expensive, the contractors converted several of their Worthington duplex pumps (with two water cylinders 8½ in. in diameter and 7-in. stroke), and drove them electrically, with a 10-h.p. motor gearing to a crank shaft at 40 rev. per min. These were very useful at East Avenue, and four of them have been at work ever since the air pressure was taken off, in the permanent pump chambers under the river. The efficiency was found to be 55%, including about 20% loss in the transmission and the motor, 19% loss in the gearing and friction, and about 6% loss by leakage.

Spoil Removal.—At the East Avenue site, after the muck was dumped into the bin, it was allowed to pass into Long Island Railroad cars and was taken out on Long Island. At Long Island City, a small part of it was handled in the same way, while the greater portion was dumped from the muck bins into 4-cu. yd. side-dumping steel cars. These were transported by a small locomotive and pulled up an incline by a Lidgerwood winch to a cantilever staging overhanging the water at the dock, where they were dumped to deck scows provided by the Railroad Company.

At times when shortage of railroad cars and scows occurred simultaneously, it became necessary, in order to keep the tunnels going, to dump the muck in the yard and reload it when scows were available.

On the Manhattan side, the muck was dumped into 4-cu. yd. steel side-dumping cars and run by electric locomotives to an overhanging cantilever staging, as at Long Island City. At this site, if scows were not available there was no alternative but to store the muck along the staging or in the tunnel. Even the muck bins at this site were too small to take care of such material.

For unloading, storing, and rehandling the cast-iron tunnel segments, eight locomotive cranes were provided. These were built by Wilson, of Liverpool. They were of the usual type, with 40-ft. booms, capable of handling 5 tons at 16 ft. radius on a 7-ft. gauge, and 1¼ tons at 35 ft. radius. They have two steam cylinders 7½ in. in diameter and a 10-in. stroke, and a drum 18 in. in diameter and 32 in. long. They can run on either 4 ft. 8½-in. or 7-ft. gauge. The gauge of the

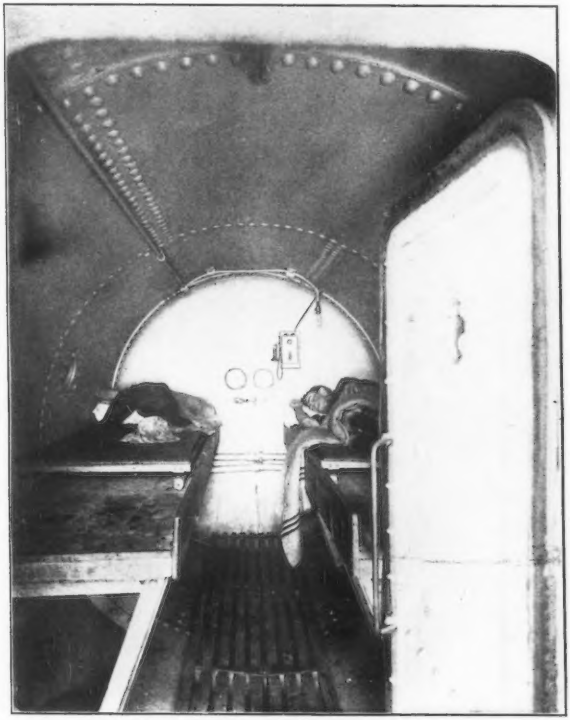


FIG. 1.—INTERIOR OF MEDICAL AIR-LOCK.

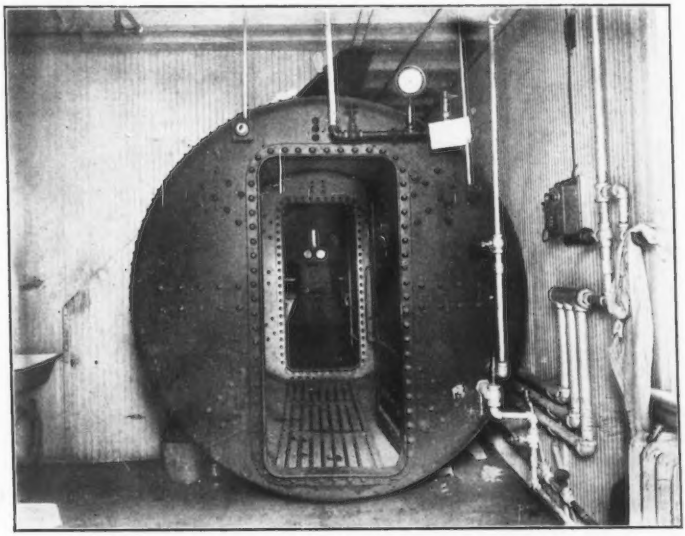


FIG. 2.—MEDICAL AIR-LOCK.

tracks on the dock was 7 ft., and a track of 4 ft. 8½-in. gauge ran down the center. This permitted the cranes, when taking a heavy lift, to have the benefit of the wider gauge; when they were running about the yard, handling light material, they stood on the 4 ft. 8½-in. gauge. One of these cranes was of 10 tons capacity, with steam cylinders 9 in. in diameter and a 10-in. stroke, running on two gauges, and capable of lifting 4 tons at 32 ft. radius, and 10 tons at 16 ft. radius.

Two cranes were fitted with a split drum for handling a two-line clam-shell, and were used for unloading crushed stone on the Long Island side. In addition to these cranes, numerous steam derricks were erected at different points.

Stone Crushers.—At each of the three sites, a No. 5, Style *K*, Gates gyratory crusher, was erected. The stone crusher was set on the ground level, the crushed stone being elevated to the screens by bucket elevators about 57 ft. high. These were 12 ft. long, 40 in. in diameter, with outside dust jackets. The refusals ran through a wooden shoot to the crusher, and the screened material went to the storage bin. The crushers were driven by a Bullock motor of 40 h.p., 220 volts, and 600 rev. per min.

The screen was divided into two sections, the first section, 60 in. long, punched with ¾-in. holes, and the second section, 84 in. long, with 1¾-in. holes. The outside dust jacket was 54 in. long, with ⅜-in. holes. They worked very well, but, for economical crushing, it would have been better to have had larger sizes, on account of the amount of sledging necessary before the stone could be passed through the crusher.

As the shields were passing through good stone at a time when little crushed stone was required for concrete, a large quantity was crushed and stored on leased land not far from the Long Island City site. Later, this was reloaded for concreting operations.

Concrete Mixers.—In the early stages of concreting the caissons and for other work, a small No. 2 Ransome concrete mixer was fixed at each site. No very elaborate arrangement was made for storage bins, as these machines were placed on the surface.

When concreting operations became possible in the tunnels, one No. 3 Ransome concrete mixer, driven by a 30-h.p. motor, was erected for each pair of tunnels. One of these was fixed in each of the Long

Island shafts, and also at the East Avenue shaft, while, on the Manhattan side, the 1-cu. yd. Smith concrete mixing plants of the United Engineering Company, including storage bins erected in the shafts, were purchased and used without alteration (Plate CXVII).

The mixers were placed high enough above the bottom of the shafts to discharge into the concrete cars, and bins of large capacity were built above them, up to the ground surface.

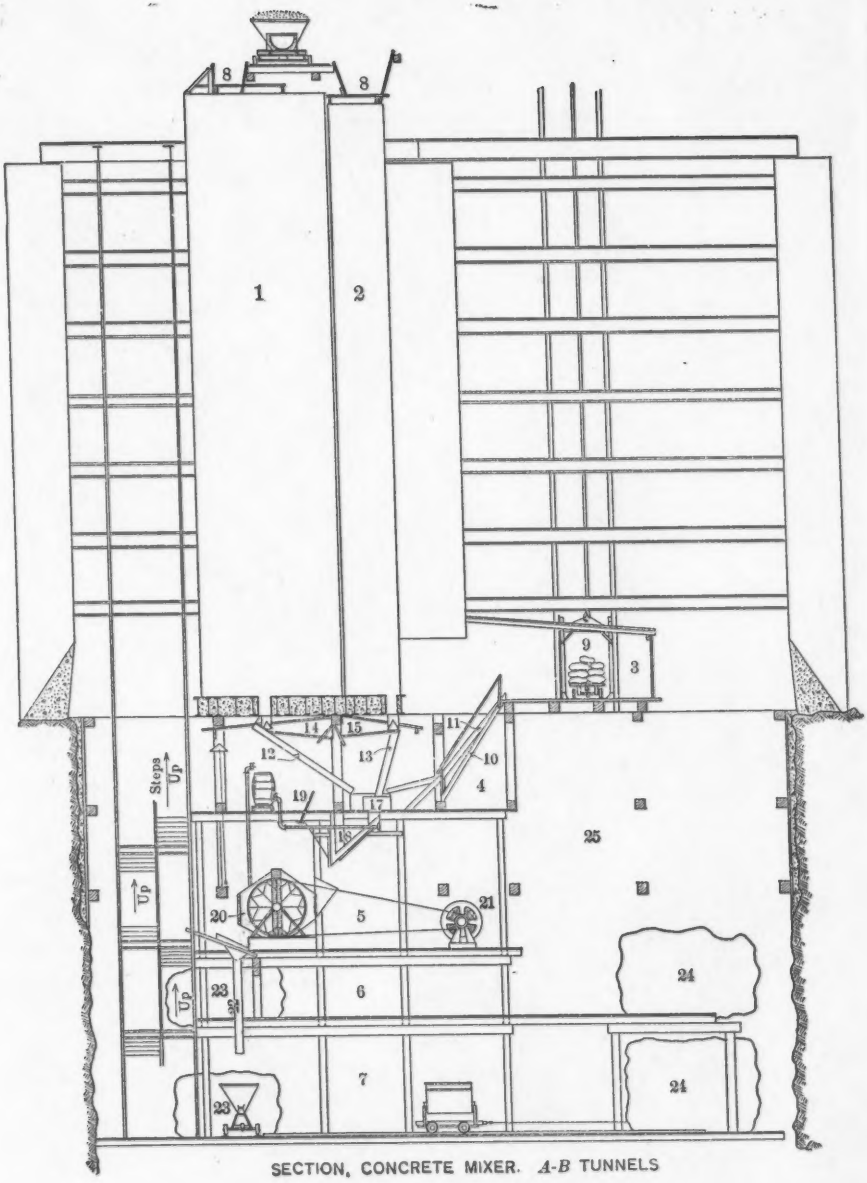
On the Long Island side, the air-tight floor was used for the bottom of the bins, and the girders from the other part of the floor were placed vertically, at 6-ft. centers, and reinforced concrete side-walls were built between them; at East Avenue the bins were made of timber. The concrete was discharged into 24-cu. yd. steel side-dumping cars, hauled by electric traction along the floor of the completed concrete lining, then hauled to an upper staging by an air-winch and incline, and distributed by mules and dumped where required. This staging was of timber, supported by byats, held on the cross-flanges of the tunnel lining. They were made in such lengths that the arch concrete could be put in without cutting the main byat, and the extra length required to reach the flanges was obtained by an extension piece on each end.

The concrete forms were entirely of timber coated with creosote. The only special feature was the method of holding up the laggings for the arch. This was done with angle bars curved to suit the arch and held by eye-bolts to the circumferential bolts of the tunnel lining. This detail is described more fully in another paper.

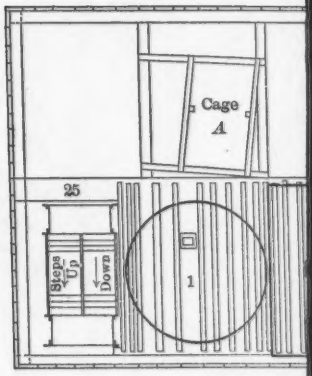
Pug Mills.—In order to prevent air leakage around the tail of the shield and between poling boards, it was necessary to have plastic clay. This was difficult to obtain, as the clay delivered on deck scows was usually in hard lumps. To soften it, two pug mills, made by C. B. Raymond and Company, were purchased, one for each side of the river. They were 9 ft. long and 30 in. in diameter, and consisted of a trough of ¼-in. steel in which a 4-in. shaft rotated 36 chilled pugging knives.

This was driven through gearing by a General Electric Company's 20-h.p. motor. Their capacity was guaranteed to be from 50 to 75 cu. yd. in 10 hours of continuous operation and feeding. These were very useful in the early part of the work, but, after a regular supply of soft Haverstraw clay was available, they were not used so often.

Dredging Plant.—In order to take care of the clay blanketing operations, a small dump-scow was purchased on which was mounted the



SECTION, CONCRETE MIXER. *A-B* TUNNELS



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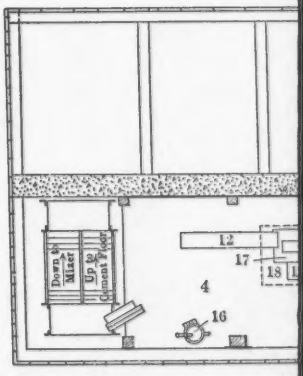

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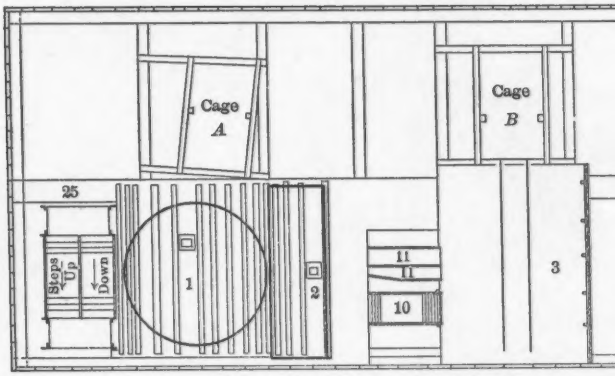
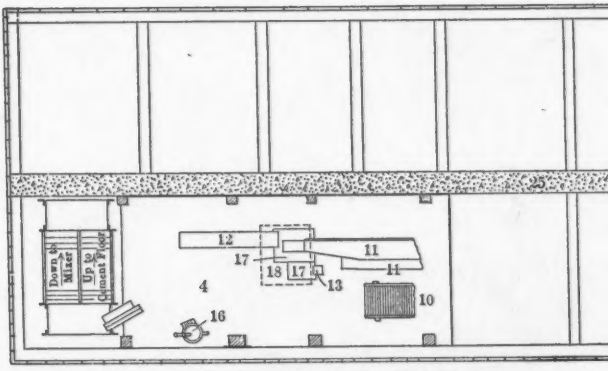
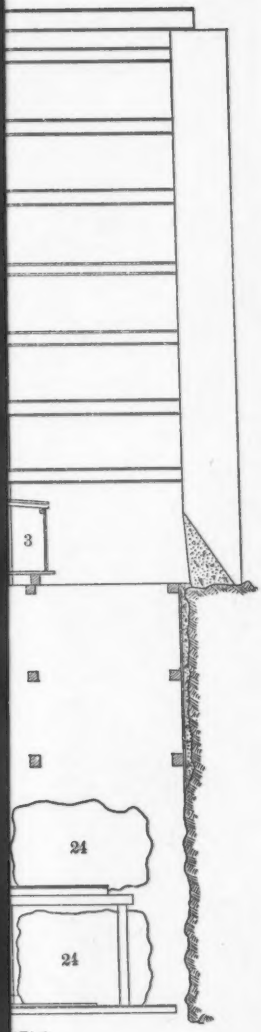


PLAN, CEMENT AND BIN FLOOR.



PLAN, HOPPER FLOOR.



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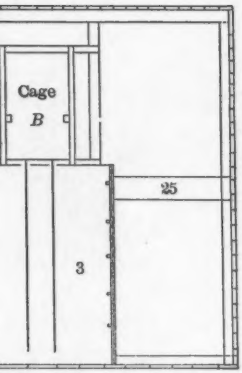
OF CONCRETING ARRANGEMENTS, MANHATTAN.



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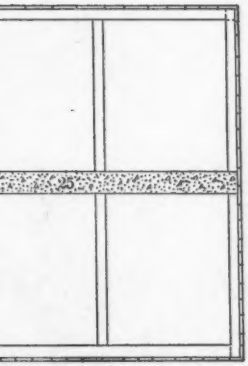
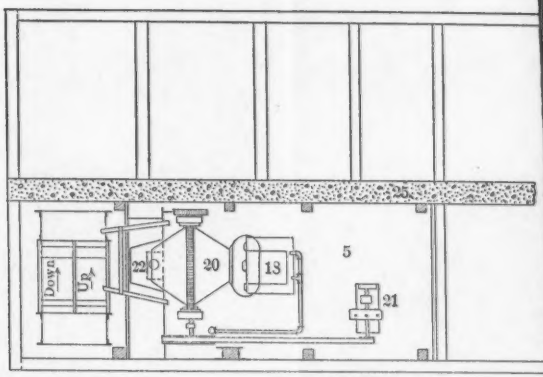
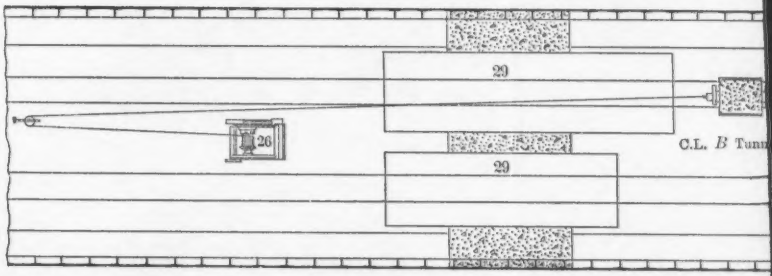
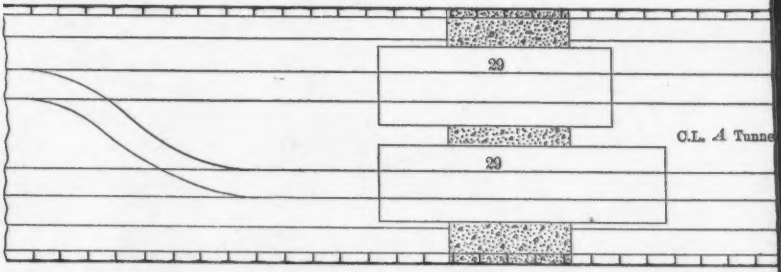




PLAN, MIXER FLOOR



PLAN AT ELEVATION OF FLOOR OF TUNNEL SHOWING TRACKS INTO TUNNE



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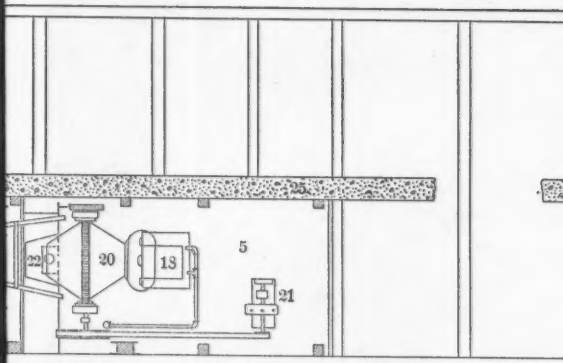
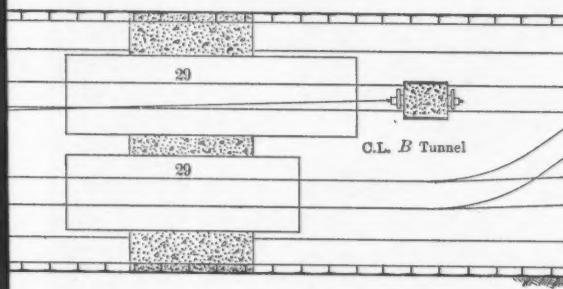
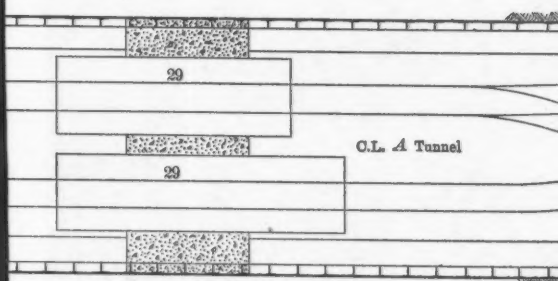
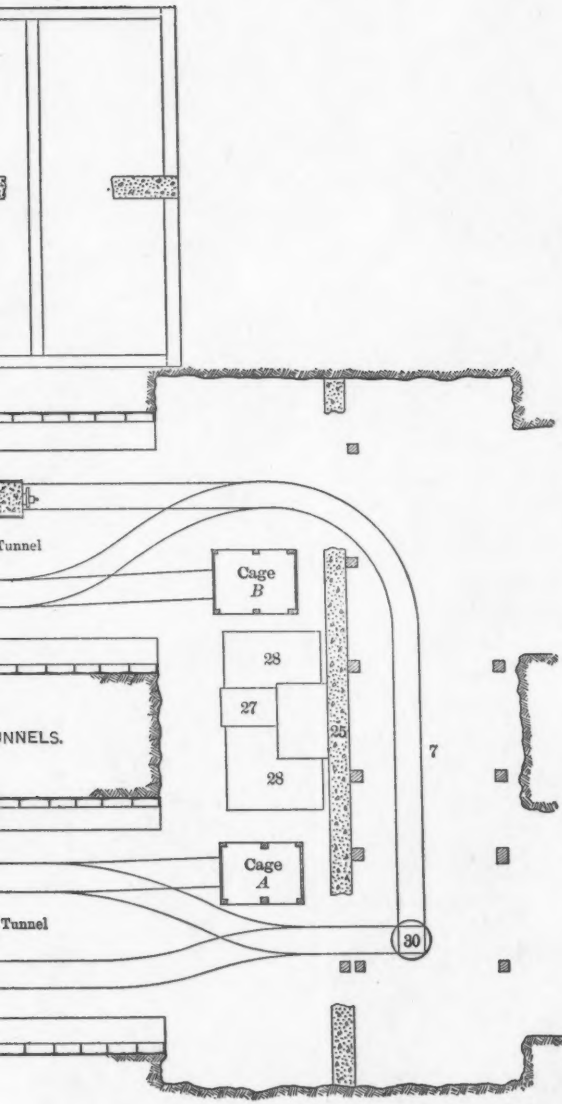


PLAN, MIXER FLOOR



FLOOR OF TUNNEL SHOWING TRACKS INTO TUNNELS.





1. Stone Bin.
2. Sand Bin.
3. Cement Storage Platform.
4. Hopper Floor.
5. Mixer Floor.
6. Old Concrete Platform used at start, for Cars.
7. Floor for loading Cars.
8. Grating over Bins.
9. Cage on Line *B*.
10. Steps from Cement Floor to Hopper Floor.
11. Cement Chute.
12. Stone Chute.
13. Sand Chute.
14. Lever to open Stone Bin.
15. Lever to open Sand Bin.
16. Water Barrel.
17. Hopper Box.
18. Hopper.
19. Lever to open Hopper.
20. Smith Mixer.
21. Motor for driving Mixer.
22. Concrete Chute to Cars.
23. Holes cut in Division Wall for *A*.
24. Holes cut in Division Wall for *B*.
25. Concrete Division Wall.
26. Compressed Air Engine for hauling Concrete.
27. Timekeepers Office.
28. Sump.
29. Material Locks.
30. Turntable.

10-ton Wilson locomotive crane with a 2-yd. digger, Fig 21. This was found very useful for digging clay stored in the slips at Long Island, for taking care of "blows," but, as the War Department limited the width of clay blankets in the river, a large dredge was necessary for re-dredging the clay blanket and depositing it ahead of the tunnels. As the prices quoted for doing this intermittent work were high, it was a paying proposition for the contractors to buy a dredge and handle this work themselves. Therefore, they purchased a clam-shell dredge 94 ft. long, 30 ft. wide and 8 ft. deep, with main engines of two cylinders 14 in. in diameter and an 18-in. stroke, driving two pinions meshing with the main 8-ft. gears. Frictions, 7 ft. 6 in. in diameter,

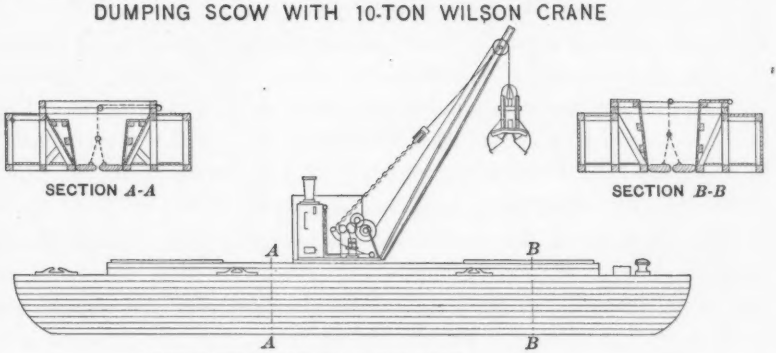


FIG. 21.

engaged in the main gears and drove two drums 2 ft. in diameter. The clam-shell bucket had a capacity of 4 cu. yd. when digging in heavy material, and 5 cu. yd. in lighter material. The weight of the 4-cu. yd. bucket loaded was 26 700 lb. Steam was supplied by a locomotive-type boiler, 5 ft. in diameter and 20 ft. long, with a grate area of 27 sq. ft., and a working pressure of 100 lb. This supplied enough steam to make a complete cycle of operations in 40 ft. of water in 40 seconds.

At the forward end, a mooring and spud-hoisting gear was fixed, driven by a steam engine having two 8-in. cylinders 9½ in. long. This geared into a countershaft which drove two anchor-rope drums, 3 ft. in

diameter, driven by 4 ft. 6-in. frictions, and two spud-hoisting drums, 2 ft. in diameter, driven by 3 ft. 6-in. frictions.

At the stern were fixed two engines, each with four single-acting cylinders, 6 by 10 in., working by a single crank directly on a worm which drove two drums 3 ft. in diameter for each winch. Five anchors, 2 500 lb. in weight, were used, and were attached by 1-in. steel ropes.

On the Manhattan side, this plant had to work in 48 ft. of water in a current varying in either direction up to 4½ miles per hour. The maximum output was 1 580 cu. yd. in a day of 10 hours.

The tugboat *Columbia* was purchased for handling the dredge and scows. Its dimensions were: 87 ft. long, 20 ft. wide, and 9 ft. deep, the gross tonnage was 96 tons. A boiler, 14 ft. long and 8 ft. 6 in. in diameter, supplied steam at 50 lb. pressure to a single-cylinder engine, 26 in. in diameter and 26-in. stroke, exhausting into a surface condenser.

Three scows were also purchased, namely, one 115 ft. in length, 32 ft. wide, and 12 ft. 10 in. deep, with six pockets, with a total capacity of 654 cu. yd., and two scows each 112 ft. long, 31 ft. 6 in. wide, and 12 ft. deep, with six pockets, with a total capacity of 588 cu. yd.

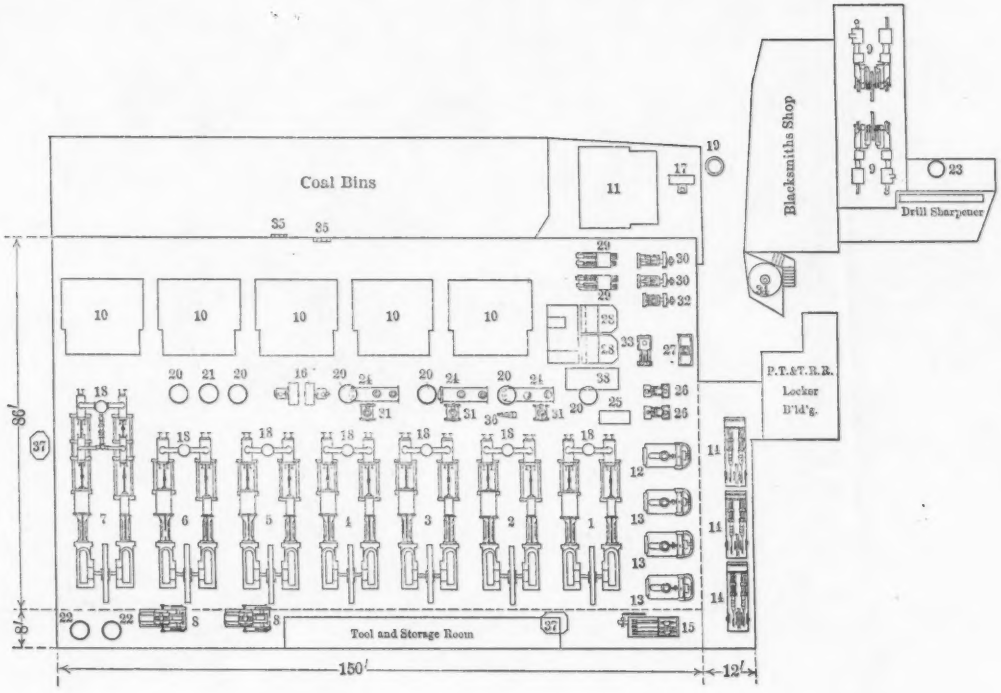
For marking off the position of the clay blankets, in accordance with the War Department's regulations, two hulks were purchased, to act as stake boats, and were moored in the river over the edge of the blanket, with large notice boards, giving particulars of the width and depth of the channel.

The foregoing is a summary of the general plant used on the East River tunnels. Plate CXI shows the general arrangement of the plant at the East Avenue site. One medical air-lock, Plate CXVI, illustrated in the writer's paper entitled "Caisson Disease and Its Prevention,"* was used at this site while compressed air work was under way, and was later shifted to Long Island City. Men's locker rooms were built near the shaft, and the plan gives a good idea of the methods of handling the materials. The powder magazine was placed at the center of the block, as far from the general public as possible. A small workshop for repairs contained the following tools:

One Sibley and Ware drilling machine with a table 16 in. in diameter and a 20-in. feed,

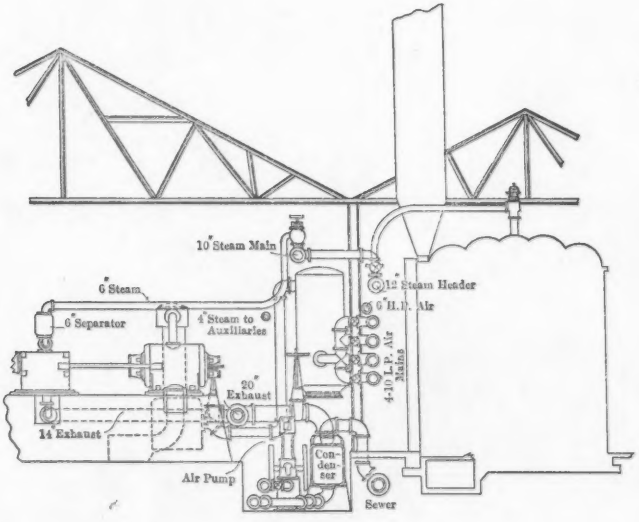
One Sagar saw bench,

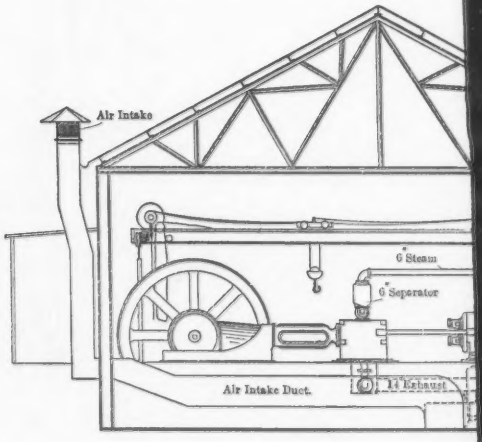
**Transactions*, Am. Soc. C. E., Vol. LXV, p. 1.

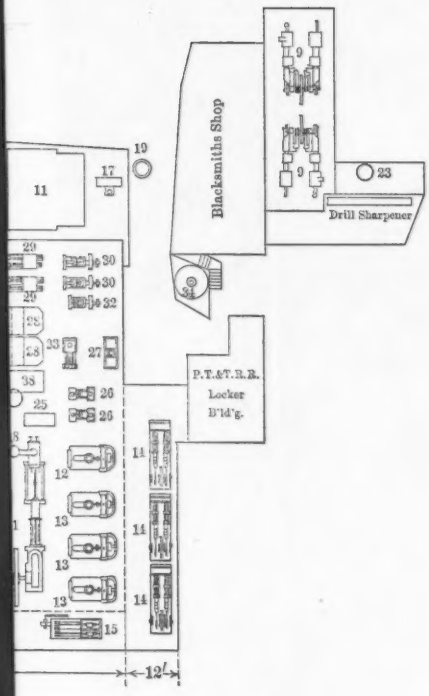


EAST R

1 Low Pressure Compressor
2 " " "
3 " " "
4 " " "
5 " " "
6 " " "
7 Combination High and Lo
8 Class *A* High Pressure Co
9 " H.C. " "
10 Stirling Boiler Original I
11 Stirling Boiler Installed i
12 160 K.W. Electric Genera
13 75 K.W. " "
14 High Pressure Hydraulic
15 Low Pressure Hydraulic
16 Blowers and Engines for
17 Blower and Engine. "
18 After Cooler for Individua
19 After Cooler for Tunnel M
20 Low Pressure Receiver.
21 Class *C* High Pressure Re
22 Class *A* " "
23 Class H.C." "
24 Condensers.
25 Hot Well.
26 Hot Well Pumps.
27 Cochran Feed Water.
28 Oil Separators.
29 Feed Water Pumps.







EAST RIV

1 Low Pressure Compressor.
2 " " "
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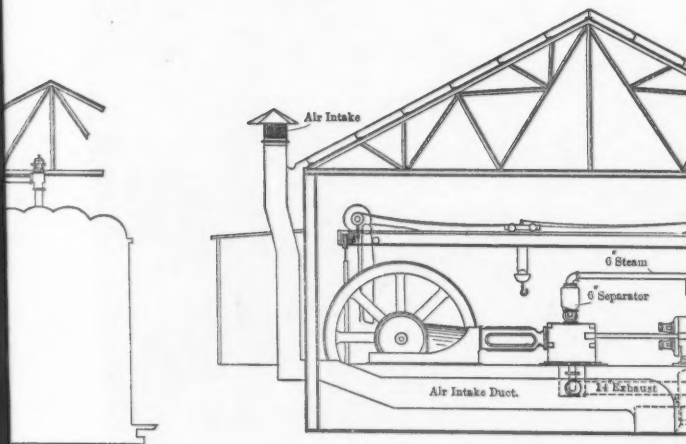
6 " " "
7 Combination High and Low P
8 Class *A* High Pressure Compr
9 " H.C. " " "
10 Stirling Boiler Original Insta
11 Stirling Boiler Installed in 19
12 160 K.W. Electric Generator.
13 75 K.W. " "
14 High Pressure Hydraulic Pu
15 Low Pressure Hydraulic Pum
16 Blowers and Engines for For
17 Blower and Engine. " "
18 After Cooler for Individual C
19 After Cooler for Tunnel Main
20 Low Pressure Receiver.
21 Class *C* High Pressure Recei
22 Class *A* " " "
23 Class H.C." " "
24 Condensers.
25 Hot Well.
26 Hot Well Pumps.
27 Cochran Feed Water
28 Oil Separators.
29 Feed Water Pumps.



POWER PLANT, EAST RIVER TUNNELS, MANHATTAN.

sure Compressor.
" "
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ion High and Low Pressure Compressor.
High Pressure Compressor.
" " "
Boiler Original Installation.
Boiler Installed in 1906.
Electric Generator.
" "
ssure Hydraulic Pumps.
ssure Hydraulic Pumps.
and Engines for Forced Draught for Original Boilers.
nd Engine. " " " " Sixth Boiler.
oler for Individual Compressors.
oler for Tunnel Mains.
ssure Receiver.
High Pressure Receiver.
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ll.
ll Pumps.
Feed Water.
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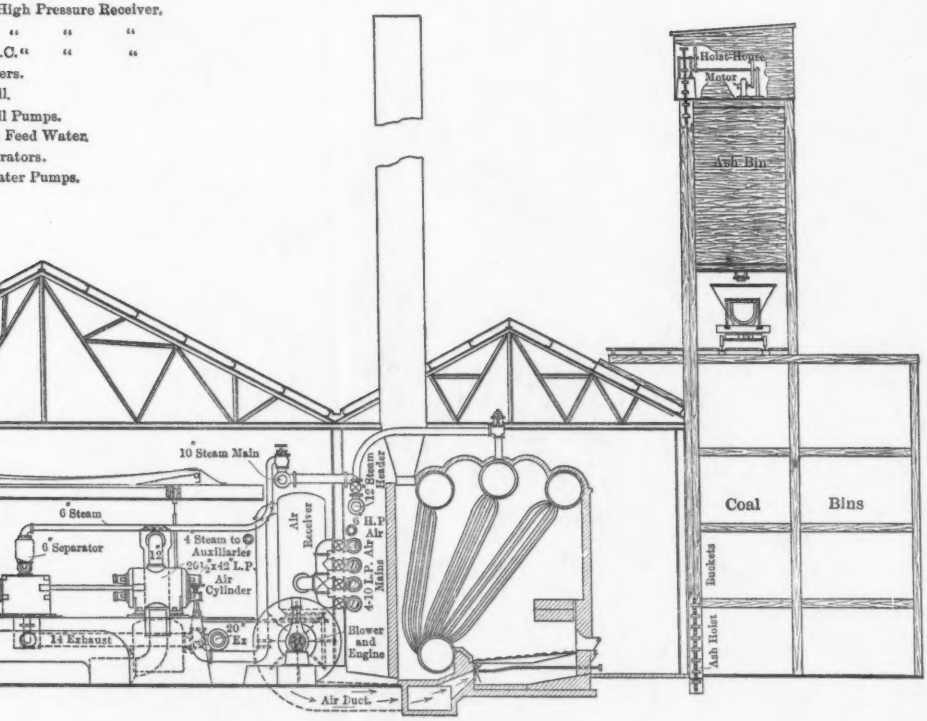
ater Pumps.

30 Pump for Condenser Circulation.
31 Edwards Air Pump.
32 Pump for Condenser Circulation.
33 Emergency Pump for Feed Water.
34 Accumulator.
35 Ash Hoists.
36 Pit Pump.
37 Air Intake.
38 Water Tank.



One 16-in. jointer and counter with 16-in. knives,
One Ajax drill sharpener for 13 ft. 6-in. drills,
Four forges.

The Long Island City site is shown on Plates CXII, CXIII, and CXIV, and has already been fairly well described, but this plan gives a good idea of the general working arrangements, showing how the materials and the spoil were handled. Two medical air-locks, Plate CXVI, were erected, along with a doctor's office at the west end of the men's dressing rooms. The powder magazine was placed at the end of the dock. An extensive workshop was provided, including the following tools:

One Sibley and Ware drilling machine, back-gear, feed, 20 in., table 16 in. in diameter,
One Fosdeck National drilling machine, 24-in. arm, drilling a 2-in. hole,
One Powis and Bale gap lathe, 18 and 36 in., and 8 ft. between centers,
One Harrington gap lathe, 28 and 48 in., compound rest, geared face-plate, 20-ft. bed, length between centers, 8 and 15 ft.,
One 24-in. Fitchburg lathe, 12-ft. bed,
One Saunders pipe machine, No. 7, complete,
Two Acme bolt-cutting machines, No. 2, complete,
One Jarecki pipe-cutting machine, No. 11, 1 to 6 in., also for bolts ½ to 2 in.,
One Williams and White double special punching and shearing machine, to shear 1-in. plate 6 ft. long, to punch 1½-in. hole in 1-in. plate,
One Bell standard guide steam hammer, with anvils and dies, 5 by 18-in. cylinder, 5 ft. 11 in. by 2 ft. 6 in. bed, height 8 ft. 2 in., anvil weight, 2 200 lb.,
One Sagar saw bench,
One band saw, No. L, wheel, 3 ft. in diameter, 500 rev. per min.,
One heavy swing saw, 30 in.,
One wood planer, 15-in. blades, 6 in. deep,
One Hoyt, No. 22, 30 by 6-in. planer and moulder, with divided rolls,
One Ajax drill sharpener, complete, with drills, dies, and dollies,

One Flather shaping machine, 14-in. stroke,
Six Buffalo, No. LO, stationary forges.

On the Manhattan side, Fig. 22 and Plates CXV, CXVII, and CXVIII, show the arrangement of the gantry and the comparatively small amount of ground available for carrying out the work. As there was little room for the storage of segments at this side, a portion of land was leased behind the boiler-house at Long Island City, where a large stock of tunnel segments was stored. These were shipped to Manhattan from time to time as required.

Permission was obtained from the Dock Department of New York City to lengthen the dock by 60 ft. This made room for two scows alongside the dock, but, owing to the Long Island Railroad rebuilding its ferry-slips and cutting off the northern side of the dock, the space available was not all that could be desired. Two medical air-locks and a doctor's office were erected near the office building. The powder magazine was placed at the end of the dock, as far from the plant as possible. A workshop under the gantry contained the following machines:

One Sibley and Ware drilling machine, back-gear, power-feed, 20 in., table 16 in. in diameter,
One Fosdeck radial drill, 2½ ft.,
One Harrington gap lathe, 28 and 48 in., compound rest, geared face-plate, 20-ft. bed, length between centers, 8 and 15 ft.,
One 24-in. Fitchburg lathe, 10-ft. bed,
One Acme bolt-cutting machine, No. 2, complete,
One Jarecki pipe-cutting machine, No. 11, 1 to 6 in., also for bolts ½ in. to 2 in.,
One Fisher punching and shearing machine, punching ¾-in. hole and shearing plate 20 in. deep,
One Bell standard guide steam hammer, with anvils and dies, 5 by 18 in. cylinder, 3 ft. 11 in. by 2 ft. 6 in. bed, height, 8 ft. 2 in., anvil weight, 2 200 lb.,
One saw bench,
One Fay and Egan, No. O, band saw, wheel diameter, 36 in.,
One heavy swing saw, 30 in.,
One Ajax drill sharpener, complete, with drills, dies, and dollies,
One Powis and Bale shaping machine, 12-in. stroke,
Four No. OH, stationary forges.

LOCATION OF PLANT, EAST RIVER TUNNELS, MANHATTAN
1 Power House
2 Coal Bins
3 Addition to Boiler Room
4 Addition to Compressor Room
5 Addition for High-Pressure Hydraulic Pumps
6 High-Pressure Compressor House
7 Oil House
8 S.P.& S.Locker Building
9 Stone Crusher
10 Lime Shed
11 Long Island R.R.Station
12 Long Island Express Shed
13 Saw Shed
14 Rope House
15 Powder House
16 Powder Office
17 Cement Shed
18 Scale Platform
19 Timekeepers Shed
20 Cement Shed
21 Cement Shed
22 Sand Bin
23 Stone Bin
24 Bolt and Drill House
25 Carpenter Shop
26 Machine Shop
27 Pipe Rack
28 Blacksmith Shop and Locker Building
29 Hydraulic Accumulator
30 Pearson's Office
31 P.T.& T.R.R. Co's Cement Shed
32 P.T.& T.R.R. Co's Locker Building
33 Consolidated Gas Co's Gas Tank
34 Cement Shed
35 Bolt House
36 Sand Pile
37 Tide Gauge
38 Resistance Box
39 Steam Pump for Air Coolers
40 Electric Pumps
41 Ring Storage
42 Lumber Pile
43 Air Cooler
44 Pug Mill
45 Radley Building
Pipe Gallery

FIG. 22.

Mr. W. De Lacy, as Mechanical Engineer, was in charge of the Manhattan plant, with Mr. W. Newall as Master-Mechanic; at Long Island City and East Avenue, Mr. W. Y. Lewis, and later Mr. E. L. Pledger, was in charge, with Mr. L. Nicholson and Mr. Joseph Varry as Master-Mechanics. Mr. A. Hjort, as Electrical Engineer, was in charge of all the electrical equipment.

Great credit is due to Mr. J. H. Jowett and Mr. A. Hoffman, of the Ingersoll Rand Company, for their original layout of the piping arrangements for the power plants, which had much to do with the economical results obtained.

The writer is indebted to Alfred Noble, Past-President, Am. Soc. C. E., Chief Engineer of the East River Division of the Pennsylvania Tunnel and Terminal Railroad, and to E. W. Moir, M. Am. Soc. C. E., Vice-President of S. Pearson and Son, Incorporated, New York, the contractors who built the tunnels, for permission to write the paper and for the use of the drawings and photographs; he is also indebted to the manufacturers of the plant who have supplied plans.

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PAPERS AND DISCUSSIONS

This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

NOTES ON THE REPLACING OF THE SUPERSTRUCTURE OF THE HARLEM SHIP CANAL BRIDGE.

By Horace J. Howe, M. Am. Soc. C. E.

To be Presented January 5th, 1910.

The highway in upper Manhattan, which was known as King's Bridge Road in Washington's time, was the only connection by land from New York City to Albany and Boston. Going north from Fort Washington, its location is easily surmised to-day, winding along the base of the foot-hills to the present Ship Canal, thence branching to the right, over Farmer's Bridge (built in 1759), and then to the left and over King's Bridge (built in 1693), to Westchester County, crossing the tidal stream early named Spuyten Duyvil Creek.

In the celebrated retreat to White Plains (1776), a dozen miles to the northeast, these bridges were useful; and the British likewise found them so, on their return from that battle ground, in their rapid march to capture Fort Washington. These operations were in a way typical of the border warfare in this vicinity during the ensuing seven years, or until the evacuation of the city (1783), and no doubt tested the bridges severely as to their loading and as arteries of traffic. King's Bridge was 24 ft. wide and Farmer's Bridge 18 ft. wide.

Happily, such conditions did not recur, and it was not until 1900 that a third bridge over the creek at Broadway came to their assistance.

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

Owing to the march of progress, all three will be taken down in the near future, and the former stream will be filled in completely.

This is due primarily to the excavation by the United States Government of what is known as the Ship Canal (1894), a waterway which connects the Harlem and the Hudson, saves distance to points on the Harlem, and enables 1 000-ton barges to cut across to the East River and avoid the dangers of the trip around the Battery. It required a swing bridge with equal waterways of 100 ft. in the clear, and at each end an approach span 100 ft. long at King's Bridge Road, or Broadway, as it is now called.

The moving of this recently built bridge and the substitution of the present one is the subject of this paper.

The change was agreed to by four interested parties, and resulted, in short, in a bridge with: Three elevated tracks for the Interborough Rapid Transit Company; two surface tracks for the Metropolitan Street Railway Company; an additional track underneath (four in all) for the New York Central and Hudson River Railroad; and a roadway increased to 35 ft. in width, and sidewalks increased to 7 ft. in width.

The City of New York, furthermore, took over the old superstructure for use at University Heights, a mile up the river.

The work was done by John B. McDonald (Contract No. 1 of February 21st, 1900), under the direction of the Board of Rapid Transit Commissioners.

Permission from the War Department was duly obtained, subject to the condition of maintaining for the new spans at all times a clear waterway, 250 ft. wide and 15 ft. deep, and a minimum clearance of 24 ft. above high water of spring tides.

Permission was obtained also from the Bridge Department of the City of New York, which considered the neighborhood carefully, and issued a permit allowing a "closing of the said Ship Canal Bridge to street travel for a total time of five days of which not more than three days shall be consecutive."

Thus the preliminaries were arranged, but direct contract work was delayed until 1905, which permitted a study of the situation in detail.

The Harlem River is an inland stream, about 6 miles long, subject to tidal influences from both ends—at the west *via* the Narrows and the Hudson River, and at the east *via* Long Island Sound and Hell

Gate. The mean rise near the bridge is 4.1 ft., and the extreme range is perhaps 10 ft. The velocity of the current at mid-tide is somewhat more than 2 miles an hour; but the tide and velocities are very variable and not to be anticipated. There is considerable summer traffic on the river.

Soundings were taken at various points, and Terry and Tench, the sub-contractors, decided to construct false-work for the new draw-bridge off shore at 216th Street (Manhattan), ½ mile above the site, 221st Street, and to construct the approach spans on shore near Broadway. It was thought that dredging would not be necessary.

Pile-driving began at 216th Street, in May, 1905. There were 15 bents of 10 piles each, and 3 bents of 15 each at the center, all being spaced carefully with relation to the floor-beams and the pivot. Besides, two clusters were driven, one at each outside end, for protection against river craft, and some others to connect with the bank. The piles were of good size, as long as 55 ft. on the deeper side, and were driven practically to refusal, penetrating from 10 to 20 ft. into the river bed, but not to bed-rock. The capping and bracing were finished in September.

South Approach Span.—Meanwhile, the framing, bracing, and decking of the bents for the south approach span was completed by August 1st, and erection begun.

The total force at this time was 50 men, using two scows (100 by 30 ft.), and a well-equipped steam lighter with a steel derrick. The riveting was completed by October 1st, and the special work for the surface tracks by October 14th. There were 14 000 field rivets.

The general scheme was to move the old span out and the new one in, simultaneously, by land and water.

The span, having been built with a panel projecting over the bulkhead, was at this time resting at that end on the cribbing of the scow, and at the other end on two sets of cribs on railroad trucks and tracks. All masonry parapets interfering with the sliding of the new span eastward to place were removed, anchor-bolts were cut, and steel wedges inserted, and a double track was lined and surfaced parallel to the bulkhead, and at its level.

On October 17th, the new span, weighing 330 tons, was moved diagonally northwest about 50 ft. to a point 115 ft. west of its final position, thereby providing room for the two scows to act in tandem. All end posts were then freed, and the span was supported at inter-

mediate panel points by a scow at the north end and the two sets of trucks at the south end.

On October 19th, at low water (5 A. M.), the old approach span, weighing 240 tons, having been cribbed up on trucks at one end, was wedged up on the scow at the other end. At 8 A. M., the north end rose clear of the pier. Some delay ensued, and it was not until 10.45 A. M. that the strain was put on the wire ropes from the drums of the stationary engine. Two pulley blocks were used in connecting with the trucks of the old span, and one three-sheave block with the new. Both were adequate, and the entire mass was moved out easily, until the old span was entirely clear of the masonry. Owing to the settlement of the tracks by this move, and to the fact that the tide had risen, the 4-in. clearance of grillage beams and abutment was reduced to scraping distance, and for a few minutes the success of the last half of the moving was a matter of conjecture. However, the span was moved in 14 min.

A temporary plank roadway was begun at once, and the first vehicle passed over the bridge at 5.45 P. M. At noon the Barber Asphalt Company began concreting the buckle-plate floor, and completed the east roadway on October 25th.

North Approach Span.—The erection of separate cribs for the new 112-ft. span was begun on September 5th, 1905, about 300 ft. east of Broadway. The trusses were at 39-ft. centers, and there were five panels, the same as in the south span; the total weight was 380 tons. Cribs were erected under alternate floor-beams, leaving one panel overhanging the bulkhead. The foundation course was composed of twelve 16 by 16-in. pieces laid side by side; the second course was four 16 by 16-in. pieces, square with the first, and the top half of each crib was made up of three 12 by 12-in. pieces of diminishing lengths. The level of the top was made 10 in. higher than the ultimate position of the bottom of the floor-beams.

Erection was begun on September 12th, and took two weeks. Then riveting began, and was finished on October 28th. There were 16 000 field rivets, practically all driven by power. Meanwhile, the New York Central and Hudson River Railroad Company completed the new north abutment.

During this time, much care was taken in lining, squaring, and leveling the trusses and floor-beams. Owing to the direction of the

shore line, it was impossible to erect the span exactly square with its final position at Broadway, and it was necessary to roll it and float it 20 ft. to the south. To do this, four tracks were laid, a pair of trucks was placed under each north post, and the weight was deposited on them. A scow, as before, took the other end, and, by the aid of a stationary engine with its lines, the move was easily made, and the two south panels extended over the water. There was not room enough for another scow, and so the weight was again dropped on the original cribs and the scow floated off. This left the span exposed to river craft.

At the next ebb tide, the second panel point was supported by a scow. Likewise, the weight at the north end was transferred at its second panel point, and all was made ready for a run westward. There were two tracks on the shore end. On November 2d the span was moved westward to near Broadway by using a stationary engine and deadman.

The next step, the track extension westward, was a puzzling problem, for the channel was only 40 ft. wide between pier and bulkhead, and the latter curved sharply to the south, west of Broadway, and it was important to insure the different clearances of the scows. Finally, a track of 200 ft. radius was laid out, carefully eased off at the approach, super-elevated and braced against the neighboring bank.

It was foreseen that the cribs would have to turn on the trucks, in a manner similar to that of a car body, and, to provide for this, two iron bearing plates, well greased, were set in place, but not fastened. The north end of the old span required three trucks, one under each chord, and the new span required two double trucks, a pair under each point of support. All walls, anchor-bolts, steel wedges, etc., were regulated as before.

On November 6th at low water (10.30 A. M.), a scow was floated under the south end of the old span, and 3 ft. west of the other scow. The clearance of the northwest corner of the pier was 8 in., and of the northeast corner 18 in.

A stationary engine had been placed 200 ft. west, and lines had been passed to the trucks of both spans and attached. The derrick-lighter was placed east of both spans and connected with a deadman west of the bridge. It was calculated that the lighter would counteract the action of the tide against the scows, and in general would preserve proper relations with the trucks.

The combined effect of the pumps and the tide caused the old span to rise clear of the pier at 2.00 P. M. An hour or so later, levels showed the new span to be an average of 10 in. above the bridge seat.

At 3.25 P. M., the signal was given to move, and in 2 min. the combined mass had moved 70 ft., and the westerly truck was at the west end of the curve. An examination of the bearing plates at the west trucks of the old span showed that the upper plates had slid along the lower ones and were nearly off; also, that the axles of the east trucks under the old span, and of all the trucks of the new span, were bending or bowing up in the middle, under the excessive strain.

This condition was partly caused by the undue pushing of the lighter. A halt was called, the cribs were jacked up, and the old span was moved separately to a clearance point, shortly after 5.00 P. M.

At 5.45 P. M. the new span was moved to place, the lines were cast off, and pumps set to work. By 7.00 P. M. the south end was at rest.

The north end was landed by using 60-ton jacks, after a hard struggle lasting until midnight. The liquid seemed to have affected the packing and caused leaks, while the tide caused much variation in the cribbing from time to time.

The bridge had been closed to vehicles from 1.30 to 10.15 P. M., and to foot passengers from 3.00 to 5.00 P. M. Planking had been laid previous to moving.

A week later, the old north approach span was loaded on two scows, and, with the help of two tugs, was floated to the south shore, west of the bridge, rolled on tracks far enough to clear the canal, and then cribbed up until required a year later.

The asphalt roadway was completed on November 17th, 1905.

The New Draw-Span.—The erection of the new draw-span began early in December, and continued until the strike of January 1st, 1906. Union labor had been getting $4.50 for 8 hours' work and wanted $5.00. The strike lasted until February 12th, when a compromise was effected. Little of importance occurred meantime. The weather was particularly fine, and later became an element of expense. Some temporary wood bracing was put into the old approach spans for protection, and odd jobs were attended to. Also, the New York Central and Hudson River Railroad began running trains under the north span.

On February 14th, power-riveting began on the roadway stringers and floor-plates of the motor platforms. The force was soon increased

to 45 men. All cross-girders had been lined and leveled some weeks before, but required constant examination. The concentration of loading had been near the middle bents, but, of course, decreased as the top work advanced. Two derrick-lighters were moved about the structure on both sides, and bumped the piles more or less; and the stationary engine and riveting guns added somewhat to the vibrations.

On February 24th, the new south span was connected with the viaduct south of it. The temporary terminal station at 221st Street was opened a month later.

Cracks having been discovered in the 27 by $\frac{7}{8}$-in. connecting angle plates of the pivot and loading girders, new ones were ordered, and arrived early in March. After much cutting out of all rivets affected, the girders were riveted up with 1-in. rivets as before. It was decided that the material was good, but having been shipped in an exposed manner, attached to the girder, the damage had occurred in transit.

The maximum force was 70 men, including 8 riveting gangs of 4 men each. In this span 122 000 field rivets and 7 000 bolts were used. By the end of March the main trusses, with sway and lateral bracing, were erected. The eye-bars had been easily connected, as the ends of the trusses had been raised $1\frac{1}{2}$ in.; but the cross-strut at the king-posts was a tighter fit.

The adjustment of the machinery in connection with the electric motors proceeded until the last of April, and from this time until the middle of June, the draw-span remained on the falsework.

Moving the Old South Approach Span.—On May 1st, 1906, the old south approach span, weighing 240 tons, was floated to University Heights on two scows, after having been run off the bulkhead on three tracks. There was some delay at one corner in detaching it from the shore crib. The contractors ordered fresh jacks, and the crib timbers were cut and slashed in order to drop that end clear of its support. This was a typical operation.

It was high tide when the destination was reached, and the scows were straddling the end draw-pier. Rollers were ready on the pier, and there was no delay in working the span westward to the temporary cribbing on the bulkhead; whence, a few days later, it was rolled still farther west to the final position on three well-braced columns.

Work at the Old Draw-Span.—The first thing done in the work at the old draw-span was the chocking of the eye-bars of the top chord

in order to provide for compressive strains while in transit to University Heights. This was early in May, 1906. As the new draw was to rest on a center pivot, and was to have only a few balance wheels or "tippers" on the outside rim, it was necessary to dress the masonry down, and furnish filler plates to take the new casting (8 ft. 6 in. in diameter), and transfer the weight to the pier; but, as the old rack was to gear with two pinions with vertical shafts, as before, it was important that the center of the rack and the center of the pivot should correspond.

Precise measurements were accordingly taken inside the old drum and resulted in finding and referencing the true point. This was then transferred by transit sidewise, up over the drum and thence to the roadway, where it became the governing point in lining up the spans.

Late in May, the old anchor-bolts were cut, and the center-pin and casting were suspended, together with the radiating rods and struts. The old rack was left in place, and a new one was put in at University Heights. One pinion was disconnected.

The old wooden trestle, used as an oiling platform, was torn down, as it was designed to oil the new shafting while the draw was closed.

Four submarine armored cables were laid across the channels in a trench, up the center pier, and through drilled holes in the coping. Meantime, electric motors had been installed at the 216th Street site, with a view to the immediate operation of the bridge on its arrival at Broadway. All riveting had been completed, and the compressors had been removed.

As it was necessary to use four scows to carry the draw, some of the piles of the platform had to be removed. This left two bents at each end and five bents under the center, supporting the entire weight of the draw and the platform. The removal of the piles was completed on May 22d. Levels, taken two days later, showed that there had been no settlement. The total settlement for the year, however, had been 3 in.

On the new draw as well as the old, the true center of the pivot was carried up precisely, and located on the roadway. Measurement to the curved end faces showed a discrepancy of $\frac{1}{4}$ in., an error not at all serious in its relation to the clearances of the approach spans.

Temporary plank sidewalks and half of the roadway were laid.

In the early part of June, as the days went by and the impending move was not made, it was reported that the draw was creeping south-

ward on its falsework, now much reduced, as stated. Observation showed that some of the piles were out of plumb, and that the caps were sagging on the center bents. Fixed lines by instrument were established on shore, and sights were taken at regular intervals for the next week or more; but only temperature variations were to be seen. The average load on each pile was 12 tons.

There was no disposition to delay the work, it may be imagined, particularly as the center pier at University Heights was completed; and operations were prosecuted with great energy by Superintendent J. F. Sullivan and his force of 50 men, up to June 14th, when the old draw was moved up the river to its present location. On that day at low water (10.15 A. M.) it was swung for the last time at Broadway through an angle of 35°, lining up northeast and southwest, and just clearing the approaches. Two scows with well-braced cribbing were floated under each arm and placed parallel to the channel, after allowing for the proper clearances at 207th Street. The suspended drum, etc., was braced and guyed to the adjoining scows. The weight was 900 tons, exclusive of cribbing, and the displacement of each scow was rated at 500 tons.

The scows had 4 ft. of water inside, and after the wedging and blocking up had been done, pumping was commenced by the aid of steam from four tugs, two lighters, and a boiler-scow. There were three 2-in. streams from the north and south scows, and two streams and an 8-in. centrifugal, from the other scows.

12.30 P. M.—The rim wheels were barely off the circular tread, and the tide had risen nearly 2 ft., as noted on the neighboring tide gauge of the Dock Department.

1.45 P. M.—The scows were 3 ft. out of water, and the depth of water in the hold was 1½ ft.

2.15 P. M.—The lower struts or arms to the rack (which had been sawed off) were clear, except at the west side, where much wedging and jacking was being done.

2.35 P. M.—All clear; the bridge drifted eastward, grazing first one approach and then the other.

2.45 P. M.—The lower struts cleared the southeast corner of the circular rack, and the tugs signaled "Go ahead." The bridgemen, being no longer needed at jacks, lines, etc., were relieved temporarily. Two tugs (each rated at 400 h.p.) were placed in front and two in the rear, and they were shifted about in going around the bend and approaching their destination. Full control was maintained through-

out the passage, the only incident being the considerable amount of mud stirred up at one point by one of the tugs. The writer was stationed on one of the middle scows. There was no creaking of the top chords, or other indication of strain in the main trusses. Some of the braces about the drum looked well "tuned-up," while the diagonals were taut. The cribs were solid and well connected.

3.45 P. M.—Arriving at University Heights, the various clearances were found to be sufficient. No bumping against the timber protection pier occurred. The diagonals to the drum slid past the round coping, and the rim wheels were well up over the rack, and, in fact, afterward, they raised somewhat. The pumps were again applied, and the tide soon helped to drop the mass on the pier. As the pivot casting drew near to place, the holes were seen to match, and the anchor-bolts were dropped in.

10.30 P. M.—The pinion engaged the rack, and, steam having been maintained all the way up from Broadway, the span was turned one-quarter around, and left in the open position above the pier protection.

Moving the New Draw-Bridge.—On the following morning (June 15th), the scows were taken to 216th Street and placed diagonally as before. The day was spent in adjusting the cribbing, calculating on the tides, as usual.

A finder-ring for the new pivot casting was placed accurately on the old pier, and wedge plates to relieve the pivot, were provided on its east and west sides. At the end piers, the latch girders and wedge plates were centered and trued up generally.

During the night rain began to fall and continued throughout June 16th, with the wind in the east. This had the effect of raising the low-water level 1.3 ft., and caused much delay in cribbing up. Later, the north scow grounded, in spite of this, showing the need of dredging, and causing further delay; so that, with high tide due by almanac at 6.40 P. M., it was 1.00 P. M. before pumping out began.

The plant was the same as before, with an added centrifugal pump and a number of hand-pumps. All were needed, for it was not until 4.15 P. M. that the pivot was clear, although the ends had been so for half an hour.

5.20 P. M.—The pivot was 10 in. clear of its seat, the latter being at Elevation 10.94 above mean high water. The top of the pier at Broadway was nearly the same (Elevation 10.89); but the wheel tread was about 1 ft. higher (Elevation 12.11), and this governed the lift at 216th Street, and the consequent time of starting.

5.55 P. M.—The pivot was 14 in. up, and there was not much more time, as the tide seemed to have turned. Tidal records showed that the flood tide was 0.4 ft. higher than usual. The order was finally given, and the immense structure (weighing with cribbing nearly 2 000 tons) was slowly pulled out into the stream, pivoting on the northwest corner and lining up, broadside to the north. The channel was none too wide, and there was some sweeping away of blocking into the river. One of the four tugs was detailed to butt out that corner, and thenceforth there was no stop or hindrance until near Broadway, where speed was reduced, lines were cast off, and the whole mass, grazing the approaches, was eased into its berth. The pivot casting slid over the rack with a margin of an inch or two.

6.35 P. M.—The casting, somewhat out of level, hung over the pier 15 in. clear, and there was ample clearance at the approaches. Lines were made fast, and the tugs were used, as before, to furnish steam for pumping water into the scows. The two scows at the north end were loaded heavily, the water being within 18 in. of the deck.

7.00 P. M.—Electric connections had been made, and lights were turned on below and above.

7.45 P. M.—The north pinion engaged the rack, and, later, closed the draw. The south pinion was evidently about right.

8.15 P. M.—Owing to the failure of a centrifugal pump, one of the northerly scows had to be sunk, and the pivot casting was 4 in. above the pier on the north side, and 6 in. on the south side, an inequality which was magnified greatly at the ends of the draw.

9.30 P. M.—The span rested on the center pier.

June 17th, 1906, Sunday:

12.30 A. M.—The scows had dropped down, and were free from the south arm. An hour later, the same was true for the north arm.

3.45 A. M.—The bridge was swung by hand-levers, and the end wedges were in place, but not adjusted.

5.20 A. M.—The span was swung half around by electric power, and it was found that it could be operated quickly and satisfactorily. The temporary wooden aprons were repaired, and the roadway was opened.

9.30 A. M.—Measurements at 68° Fahr. showed that the space between the draw and the approach span on the north side measured $2\frac{5}{8}$ in. and on the south side $2\frac{3}{4}$ in.

The next week was spent in shipping away timber, thereby releasing the scows, removing the chocks from the top chords, taking down the pile platform, adjusting the aprons, wedges, and pinions, installing the two circular feeder tracks at the pivot, and connecting with the third rails above. By the end of June, the approach spans had been lined up, and the rocker posts plumbed up. The asphalting was also well advanced.

It was decided to put in fixed aprons on the approach spans, and to close the span in one position only (without reversing), on account of the number of important rail connections. Later, it was found necessary to stiffen the rack with iron knees, at intervals.

Much of the new equipment was done by the Bridge Department of New York City, in accordance with its practice on the Harlem.

This is the only pivot-bearing draw on the river. The weight on the pivot is 1 500 tons. It is operated by two 60-h.p. electric motors, and, considering its weight, is handled more easily and quickly than any other. Occasionally, it delays traffic, while opening and closing, for a period of only 4 min. In addition to signals by whistle, and electric bells, there is a telephone connection (seldom used) between tower and engine-room.

During 1907 and 1908, the shafting was partially renewed several times, due to causes which were exceedingly difficult to locate, but resulting in twisting the shafting. In June, 1907, the latch girders were firmly fixed by reinforced concrete around and about them for the entire distance between the pedestals, and thus the play at the ends of the draw was much reduced.

The contract price for removing the old bridge and replacing it with the new one was about $250 000.

Old North Approach Span.—In November, 1907, the old north approach span, which had been left on the south bank, was rolled toward the river and received by scows, in much the same manner as described previously. The scows floated broadside through the draw channel, and were afterward swung around, with one tug in front and one in rear. The top of the east pier at University Heights was over-run comfortably, and the rollers did the rest. As no water was pumped into the scows, several tides were necessary before the final position on the masonry was reached (November 29th, 1907).

University Heights Bridge was declared open by Mayor George B. McClellan on January 8th, 1908.

In all these operations, extending over a period of two years, there was no accident of a serious character to any of the force employed.

The Rapid Transit Commissioners were represented by George S. Rice, M. Am. Soc. C. E., Chief Engineer, Alfred Craven, M. Am. Soc. C. E., Deputy Chief Engineer, Sverre Dahm, M. Am. Soc. C. E., General Inspector of Designs, and by C. V. V. Powers, M. Am. Soc. C. E.,

Division Engineer. The writer acted as Resident Engineer, and takes this occasion to make grateful mention of his subordinates, who continuously gave their best services. They were, from time to time, as follows:

Messrs. R. A. Berry, L. C. Devery, J. D. P. Hogue, R. J. Smyth, C. M. Torpey, and W. P. Vallely, Assistant Engineers; W. C. Martin, Rodman, and Theodore Belzner, Jun. Am. Soc. C. E., Inspector.

The Rapid Transit Construction Company was represented by George H. Pegram, M. Am. Soc. C. E., Chief Engineer, and A. S. F. Berquist, M. Am. Soc. C. E., Inspector of Steel.

Historical Notes.

In searching for records of similar operations, the first, chronologically, of any magnitude, is seen to be the first in magnitude with reference to the tonnage moved, if not in regard to the velocity of the current and the exposure to gales; and, later, in relation to special appliances used in raising to place. This was the work of Robert Stephenson in connection with the Conway and Britannia Bridges, as recorded chiefly by his assistant, Mr. Edwin Clark.*

Mr. Stephenson had led up to those pioneer monuments by various achievements in his practice. He had acquired habits of close technical experimentation, and of co-operation with other engineers, architects, parliamentary committees, boards of directors, and, in fact, with men of all classes. He could say the right word at a dinner; as for instance when he noted the reduction of time between London and Dublin made possible by the bridges, and spoke of engineers as being instrumental in bringing about a better understanding between the English and the Irish.

It required a very able man to make the jump to the first long-span wrought-iron bridge; for, as Mr. Clark says, the process of uniting wrought-iron plates by rivets had been hitherto a peculiar feature of boiler-work or iron ship-building and had been little resorted to by the Civil Engineer, until Mr. Stephenson proposed the construction of these bridges.

All wonderful things cease to become so, however, when the separate steps are studied; and perhaps the transition from an iron ship, with

* "The Conway and Britannia Tubular Bridges," London, 1850, by Edwin Clark, Resident Engineer.

ribs and knees at short distances, to the rectangular tube, with web stiffeners and gussets every 2 ft., together with the cellular top and bottom, borrowed from a previous design of his, was not intrinsically so great an innovation, after all.

The spectacle of an accident to an iron vessel and the evidence of its stanchness in resisting the waves, and holding together, is said to have exerted a strong influence on Mr. Fairbairn, the manufacturer, and through him, on Mr. Stephenson, as to the ability of a similar iron framework to carry rolling loads.

Later, when the method of floating the tubes was considered, there naturally followed the idea of doing without the use of scows, but, for good reasons, that method was not adopted.

The Conway Bridge.—Work was wisely begun on the smaller bridge, at the Conway River (May, 1846), and, after two years of experiment and construction, the first tube was floated from its falsework, some 600 ft., to the site it now occupies.

Steam riveting machines had been used throughout, and half-round "rimmers." The type of tube had been decided chiefly from tests on a model of one-sixth the size of the bridge. Not satisfied with this, Mr. Stephenson supported the tube at its ends, cut out the falsework, and loaded it with cars containing iron plates. The resulting deflection was so small that no similar test was necessary again.

The floating of the tube (26 ft. high, 14 ft. wide, inside, 412 ft. long, and having a weight of 1 417 tons of 2 000 lb.) was done by six scows (each 98 ft. long, 25 ft. wide, and 8 ft. deep) arranged in two sets of three. This provided for an overhang at each end, sufficient to take the timber seats prepared in the recesses of the abutments, at some distance from the shore line. The tubes were 6 ft. 6 in. above the water.

There were two guide-chains under water, with which it was supposed that the movement could be controlled in a manner similar to that of a ferry on a suspended cable, but they proved almost useless, and most of the hauling was done by two Manila hemp cables, each operated from a 50-man capstan. The force was about 350 men, on land and water. A steamer was at command, but seems to have acted only as a utility boat.

Some progress was made against the rising tide, but more when it slackened at the flood, and enabled the tube to be easily hauled to place, clearing the abutments by 4 in. at each end. The average rise

of the tide was 18 ft. and the clearance of the bridge was also 18 ft., so that a lift of about 12 ft. remained to be made.

This was done by a hydraulic press (18-in. ram with a 6-ft. stroke) at each end.

The weight of the tube was transferred first to a cast-iron frame inside; thence to eye-bars with pin connections; and thence to the cross-head of the press above. A small steam engine and pump, to force water into the press, were attached.

Barring the pulling out of a cylinder head, and the delay in replacing it with a stronger one, the work was continuous; but cast iron was either eliminated or heavily reinforced with wrought iron at the Britannia Bridge.

Mr. Stephenson passed through on the first locomotive in April, 1848.

It may be well to note here the early decision with regard to painting plates before riveting them together, which was, that close contact is thereby prevented. The claim was made that old pieces of riveted work were most sound at the joints. Where necessary, a mixture of red and white lead was used as a filler.

The cellular floor and top chord, built up of plates and angles, were large enough to allow a man to paint them from the inside. The track floor was cheaply water-proofed with a mixture of coal-tar, lime, and turpentine. A corrugated, galvanized-iron roof was constructed over the whole tube.

The price per (long) ton was £37, and the total cost of the Conway Bridge was £145 200.

The Britannia Bridge.—The Britannia Bridge connects Wales and the Island of Anglesey, across Menai Straits. The two main spans are each 460 ft. in the clear, and were floated to place, whereas the approach spans were constructed on falsework at the site.

The Government required 105 ft. clearance above high water, and therefore a hollow, center tower, 220 ft. high or more, was required on Britannia Rock. The foundations were begun on Good Friday, 1846, and the last stone was laid in June, 1849, the masons working 12 hours a day for 6 days in the week, in order not to hold back the tubes. The latter were 30 ft. high, 14 ft. wide, inside, and weighed 2 150 tons.

In floating each tube 650 men were employed, about 400 being

sailors hired for the occasion. Eight scows were used, in much the same way as at the Conway Bridge, and were guided by Manila cables buoyed up by casks. Tarred Manila ropes were used for hauling, with "cable-stoppers" and "messengers."

The four tubes were handled in conformity with written instructions, after Mr. Stephenson had made experiments with models on a small pond. The various rocks and shoals were indicated, together with the scows, tubes, and lines, and everything that it was possible to foresee was discussed.

At the moving, he and others (including Captain Claxton, who had immediate charge) occupied a position on the top of the tube, and from that elevation signals by flags were given to each boat concerned.

Although, in floating the four tubes, there were instances of heavy cables snapping, and of severe bumps, yet, in general, the men were equal to the emergencies, and landed the tube each time in the recesses of the masonry, before the tide fell. Two steamers were in attendance.

The lifting machinery was considerably improved, the press having a 20-in. ram, lined with brass, and having a 6-ft. stroke as before. Leaks were stopped with oatmeal gruel and sal-ammoniac, and, later, by a second U-shaped leather collar below the first.

Brickwork in cement followed closely, but, in spite of this, the first tube, during one lift, dropped 8 or 9 in., doing considerable damage.

In the raising of each tube, 6 ft. per day was accomplished for about 16 days, and a single track was put in use in March, 1850. The fourth tube was floated in July, 1850.

The Britannia Bridge was riveted by hand. The total cost was £601 865, and of this, one-fourth was for masonry. Both the Conway and Britannia Bridges are still in use.

Associated with Mr. Stephenson were Mr. I. K. Brunel, and Captain Claxton; and as these gentlemen were soon afterward engaged at the Chepstow and Saltash Bridges, in similar movements, brief mention of these may naturally be made next.

The Chepstow Bridge.—The chief feature at the Chepstow Bridge (over the River Wye), barring the pneumatic process used in sinking the cylindrical foundations, was the design and erection of the 300-ft. channel span.

The top chord was composed of one wrought-iron tube, 9 ft. in diameter, treated like a trussed beam of three panels, except that there were two suspended chains, with the track between. The height of the truss was 50 ft. Pins 7 in. in diameter were used at the ends. A second tube for the other track enabled lateral bracing to be introduced, and stiffened each.

Following out the cautious method of the Conway Bridge, the first span was erected on falsework and tested with a live load of 2¾ (long) tons per linear foot. The wrought iron in each truss weighed 460 (long) tons. It was then taken apart, and the tube itself, supported by posts and chains, was floated (April, 1852), on six wrought-iron barges. It had to be done quickly, as the rise of spring tides was 40 ft. The use of crabs with two barrels enabled hauling-chains to be wound without difficulty.

The tube was lifted by jacks, during the day, some 40 ft., and, later, it was raised 50 ft. more, to its final position.

A single line was opened up in July, 1852. This was the first circular tube of large size introduced as a bridge member.

The Saltash Bridge.—In the Saltash Bridge, a single-track structure across the Tamar River, near Plymouth (also on the Cornwall Railway), the top chord tube was made elliptical (16 ft. 9 in. wide and 12 ft. 3 in. high). The bottom chord was double. The type of truss was a bowstring, with the string curved equally with the bow.

Mr. Brunel* (3d) states that "the mode of floating and lifting the superstructure had great influence in the preparation of the design" —meaning cylindrical foundations and cast-iron columns, as well as truss design.

There were two single spans of 455 ft., 56 ft. high, and each contained 1 200 tons of wrought iron. The bottom chord was of 7 by 1-in. bars, 20 ft. long. There was a solid ballast floor throughout.

The work was begun in 1853, but was delayed, on account of the difficulty of founding a 35-ft. cylinder, by using compressed air, at a depth of 88 ft. below high water. This was the first successful attempt on a large scale.

Meanwhile, the superstructure was manufactured, put together, and afterward tested at its falsework by a live load of 1 340 tons, uniformly distributed. The deflection was 5 or 6 in., and was deemed satisfactory.

* "The Life of Isambard Kingdom Brunel, Civil Engineer," by Isambard Brunel, B. C. L., London, 1870.

It was as late as September 1st, 1857, before the first span was moved, and information in relation to it is comparatively meager. The methods used at the Britannia Bridge were followed. The actions of the 500 men engaged were regulated in accordance with printed instructions which had been issued, and with flag signals from the "bridge," where Mr. Brunel and others were stationed during the passage. Mr. Stephenson was too ill to be there.

The move was at first outward from the shore, then, after an interval in which the hawsers were shifted, up the river, endwise to the center pier, then a quarter turn was made, and finally, an accurate placing by strong lines at the piers. There was a clearance of a few feet above the water. The tide was not swift and there were no mishaps mentioned.

The lifting was done 3 ft. at a time, separately at each end. At the center pier, cast-iron columns (braced) were built up as fast as these lifts were made; but, at the shore end, the masonry proceeded more slowly. There was no cause for hurry, as the river traffic went through the other opening. There was finally a clear distance of 100 ft. above spring tides; and the total height above the foundations was 260 ft. to the highest point of the trusses.

Three hydraulic presses or jacks were used under each end. The rams had a screw-thread, arranged with a nut in such a way that the tube was supported at all times. Timber packing was also used. By July, 1858, the full height was reached. The second span was floated successfully soon after. In May, 1859, the bridge was declared open by Prince Albert, after whom it was named the Royal Albert Bridge. The total cost was £225 000. The fiftieth anniversary has recently been celebrated, the old bridge remaining as it was originally.

Other Bridges.—Passing by well-known examples of this method of erection—the Moerdyck Bridge (1875), the new Tay Bridge (1885), the Hawkesbury Bridge (1889), and others of 500 tons or less, in which about the same or similar principles were involved—we come (1890) to the 523-ft., single-track, through bridge across the main channel of the Ohio River, below Pittsburg, on the Ohio Connecting Railway (Pittsburg, Cincinnati, Chicago, and St. Louis Railway), erected by the Keystone Bridge Company, C. L. Strobel, M. Am. Soc. C. E., Chief Engineer.

The Government, having required the long span and also a clear

height of 75 ft. above mean water level, and having limited the time during which the river might be obstructed, it became necessary to build the falsework at a corresponding height, assemble and erect the bridge upon it, and then float the structure to its site, as quickly as possible.

Nineteen bents of 5 piles each were driven and cut off 16 ft. above the water. On these were framed bents of 5 posts each, with an 80-ft. sill and a 32-ft. cap, all with the usual railroad trestle dimensions. At the base, across each alternate bay, 20-in. I-beams were added. Under these were duly placed nine ordinary coal barges (130 by 26 by 8 ft.).

The weight of the bridge was about 900 tons. The trusses were 65 ft. high. The bracing of the bents into towers and the binding of them together by rods with turn-buckles was susceptible of careful study, and, once accomplished, eliminated the principal cause of uncertainty. At Hawkesbury, the Union Bridge Company had effected this by a single pontoon, 335 ft. long, well braced with timber and wire cables internally, but with the superstructure at a much lower level. In both cases, the hauling to place was done by cables.

At Pittsburg, two scows were fastened to the bows of the end barges (leaving the three barges in the middle unattached), and on these scows four engines controlled the lines up to the bridge piers. The nine barges were braced together at the bow, and, after floating out to clear the piles, they were braced likewise at the stern. The current in the river was slight, and, although a couple of heavy showers occurred while the scows were being moved the few hundred feet necessary, there was no bumping against shore or against obstructions.

The itinerary of the work was as follows:*

8.50 A. M., August 19th.—Sufficient water had been pumped from the barges so that the mass rested on them.

11.15 A. M.—The barges had been floated out toward the center of the river, thereby clearing the bents of piles.

7.30 P. M.—Water was again admitted to the barges, and the span cleared the bridge seats by 2 ft.

8.20 P. M.—The span was about 9 in. clear. The work was blocked up and left until the following morning.

7.30 A. M., August 20th.—The lowering was continued, so that the bridge rested on its piers. The barges were removed later in the day.

* *Engineering News*, September 20th, 1890.

There had been no distortion of bents or barges, nor any important casualty of any kind.

In Canada, the Dominion Bridge Company has erected several bridges of note, in this manner.

The Coteau Bridge (1890), across the St. Lawrence (Grand Trunk Railway) west of Montreal, has single-track spans of 175 to 223 ft.

The Shubenacadie Bridge, in Nova Scotia (1901), across an arm of the Bay of Fundy (Grand Trunk Railway), has single-track spans of 215 ft., each weighing 150 tons.

The Miramichi River Bridge, Newcastle, N. B. (1902), has single-track spans of 204 ft., each weighing 320 tons.

The French River Bridge, Ontario, a single-track span of 415 ft. weighing 1 424 tons, was erected by the Canadian Pacific Railroad in 1907. This was slid endwise from the embankment upon a scow (155 by 33 by 12 ft.) and thence was hauled to place by lines from an engine on the scow.

Local difficulties in all these cases were overcome successfully.

Sand Jacks.—The use of sand in jacking up or in letting down bridges of moderate tonnage dates back to 1878, or even earlier, in Europe.

The most noteworthy instance of this sort in America was witnessed by the writer on Sunday, December 20th, 1903, at Newark, N. J., when the 220-ft. double-deck draw-bridge of the Delaware, Lackawanna and Western Railroad, across the Passaic River, was moved, complete and ready for use, to its present position, under the direction of Lincoln Bush, M. Am. Soc. C. E., Chief Engineer. The bridge is center-bearing, and has four tracks. It was moved up stream some 40 ft. and lowered 10½ ft. from its level at the old pier.

Several years before, it had become necessary to renew the old draw, and as grade-crossing work was in progress in the vicinity, the new draw was designed accordingly, and the lower deck used temporarily.

The weight of the bridge was 1 017* tons, and the sand and timberwork 936 more, a total of 1 953 tons on the scows above the deck.

A year or so before, there had been experiments on the action of sand in compression by loading two 83-ft. girders on a pair of plungers

* *Engineering News*, Vol. 50, p. 576.

fitting into 4 by 6-ft. boxes. It was found that there was little swelling, that the most favorable angle of the side holes was 45°, and that the holes should be made funnel-shaped, with a diameter of 2 in. inside and 3 in. outside the box. There were also 2-in. holes in the bottom of the box. All holes were regulated by slides.

The bridge rested on two scows under each arm, and there were four sand boxes of 12 by 12-in. stuff, each 52 ft. long, 12½ ft. high and about 4 ft. wide, inside measurements. The plungers had a clearance of ½ in., and their pressure on the sand was less than 1½ tons per sq. ft.

The rise of tide at Newark is generally somewhat more than 4 ft. but on the day the bridge was moved the wind and tide were perverse, and, unfortunately, there was a rain all day; so that it was with difficulty that the sand was kept in prime flowing condition, tarpaulins being used to protect it as much as possible.

In spite of all precautions, there was irregular settling, and time was lost in balancing the movement. The bridge had been raised by the tide, helped by four centrifugal pumps. A boiler scow and pump was held in reserve. The rate of movement was as follows:

At 6.30 A. M., the old supports were cleared,
" 9.35 A. M., began lowering at the new site,
" 5.00 P. M., the span rested on the new pivot,
" 6.15 P. M., all scows had been released,
" 8.30 P. M., the first train crossed over.

The objection to this method is the likelihood of wetting the sand (which in this case was screened through a sieve of ¼-in. mesh) and the consequent ignorance as to what is going on inside the boxes. Under such conditions, looking for evidence of strain in heavy timber is a responsibility which few engineers would care to take.

Finally, other flotation operations on the Harlem River should be mentioned; namely: At Macomb's Dam Bridge, at Madison Avenue, and possibly at the crossing of the New York Central and Hudson River Railroad—operations which deserve to be recorded in detail.

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* *Engineering News*, Vol. 50, p. 576.

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AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

CHARACTERISTICS OF MODERN HYDRAULIC TURBINES.

Discussion.*

BY MESSRS. LEWIS F. MOODY AND JOHN C. PARKER.

Mr. Moody. LEWIS F. MOODY, ESQ.† (by letter).—This may be considered the most noteworthy contribution on the subject since the late R. H. Thurston, M. Am. Soc. C. E., presented his paper on "The Systematic Testing of Turbine Water-Wheels in the United States." This paper was presented before the American Society of Mechanical Engineers in 1887.‡ The turbine tests reported by Professor Thurston, like those reported by Mr. Larner, were made at the Holyoke flume, and were notable for the efficiencies developed, the highest which had been obtained in reliable tests up to that time. The tests were made in 1883.§

The wheel tested by Professor Thurston was a 36-in. R. H. "Hercules" turbine, built by the Holyoke Machine Company, and the maximum efficiency, on each of the three tests made by him, was 86.94%, 86.18%, and 87.08%, respectively (Tables IV, V, and VI). The specific speed corresponding to the point of maximum efficiency is 196, in the metric system.

Since the date of Professor Thurston's paper, great progress has been made in water-wheel design, both in America and in Europe. Mr. Larner's paper, and the remarkable results reported by him, indicate the recent advances which have been made in raising both the efficiency

* This discussion (of the paper by Chester W. Larner, Esq., printed in *Proceedings* for August, 1909, and presented at the meeting of October 20th, 1909), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Assistant Professor of Mechanical Engineering, Rensselaer Polytechnic Institute.

‡ *Transactions*, Am. Soc. Mech. Engrs., Vol. VIII, p. 359.

§ Bodmer, "Hydraulic Motors," pp. 401, 481–490.

and speed of turbines. Mr. Larner's wheels have shown specific speeds as high as 450 (in the metric) or 100, in the foot-pound system, with an efficiency of practically 80 per cent. Mr. Moody.

As efficiency and specific speed are the most important issues raised in Mr. Larner's paper, the writer has endeavored to show how they are related, and also how various turbines compare in regard to these two factors, by constructing a curve, Plate CXIX, with specific speed for the scale of abscissas, and efficiencies for the ordinates, using the results of all available tests showing the highest known efficiencies. This curve furnishes a basis for comparing the performance of any turbine with the best previous results shown by turbines of the same specific speed.

In plotting it, the speed, head, and delivered horse-power at the point of highest efficiency of a given wheel have been used in calculating the specific speed. That is, a wheel, when running at a gate opening in excess of that giving maximum efficiency, and at a speed in excess of the normal speed, may develop a much higher specific speed than that plotted; but, for the purpose of comparison, the specific speed corresponding to the run on which the efficiency was developed, has been used in plotting the point. On wheels developing unusually high speed, additional points have, in some cases, been plotted, giving the efficiencies corresponding to higher specific speeds for the same wheel, and showing by a short curve how the efficiency drops on a given wheel as the specific speed is increased beyond the normal.

Thus, points 24 and 27 on Plate CXIX both correspond to the same wheel, a 56-in. Samson turbine, point 24 corresponding with the maximum efficiency shown by the test, and point 27 showing that with a much greater specific speed the efficiency dropped somewhat but not greatly in this case (this curve is indicated by a dash and two dots). Other tests of various Samson turbines also give points lying on this same curve, indicating similarity of design.

In the same way, a Holyoke test (No. 1778), reported by the Allis-Chalmers Company, has been plotted at several points, giving the curve shown by a fine line; and the test of a Swedish turbine (built by the Karlstad mek. Verkstad, Filiale in Kristinehamn, Sweden),* is given by a dotted line, *H-I*.

It will also be noticed that the highest curve given (a solid, heavy line) passes through two points of Mr. Larner's wheel No. 1796. This indicates that, while no turbine is known (to the writer, at least) which exceeds in efficiency the results shown by this test, No. 1796, at the speeds corresponding to it, still, the high-speed point ($n_s = 446$) was not obtained at the point of maximum efficiency of this wheel; and it would seem probable that a wheel could be designed for this particular specific speed which would exceed this value of the efficiency. (The

* *Zeitschrift für das gesamte Turbinenwesen*, Heft 3, V Jahrgang, Jan. 30th, 1908.

Mr. Moody. specific speeds, n_s, are in the metric system throughout, in order to adhere to an international standard.) The equivalent "unit speeds," to use Mr. Larner's term, or "type characteristics," Mr. Zowski's term—both meaning specific speed converted to the foot-pound system of measures—are given at the top of the diagram.*

In order to show the recent progress of water-wheel design, and to compare American and foreign achievements in this branch of engineering activity, a curve constructed by Messrs. V. Graf and D. Thoma,† has been reproduced by dots and dashes. This curve was at that time of the same nature, and was intended to show the same things as the writer's full-line curve on Plate CXIX. That is, Graf and Thoma intended their curve to represent the best known results, and up to a specific speed of 200, it represented very fairly the state of the art at that time. Above that speed, however, their curve fell considerably below many results which had been obtained. The reason for this was that many German engineers did not then, and do not now, accept Holyoke tests as correct, and are inclined to ignore all results obtained in America. There were many American tests considerably exceeding the curve of Graf and Thoma in efficiency and speed; but, as has been said, very few available results have approached the new curve plotted by the writer on the basis of Mr. Larner's tests and some other recent results.

Since the international aspect of the question under discussion naturally presents itself, the curves furnished by Professor Ernst Reichel,‡ have been indicated by curves of dashes at *D-E* and *F-G*. As stated in the article mentioned, the curve, *D-E*, represents present German accomplishments, and shows the progress that has been made in Germany since the Graf and Thoma article was written. The second curve, *F-G*, is from tests of an experimental turbine designed by Wagenbach, and indicates the highest specific speed thus far attained in Germany. The efficiencies are below those given in Mr. Larner's paper by from 3 to 4 per cent.

In constructing the curve for maximum attained efficiencies, the writer, besides plotting all the tests known to him in printed form, wrote to eight of the leading turbine manufacturers in the United States for their most recent and highest results; and, although he has not yet heard from all of them, he believes that the data thus collected represent very fairly the best American practice to-day. With only two exceptions, the manufacturers have responded most liberally to this request, and, while the writer is not able to assume responsibility for the accuracy of the data, he has been furnished with signed reports of the tests, and believes them to be authentic, with few if any exceptions.

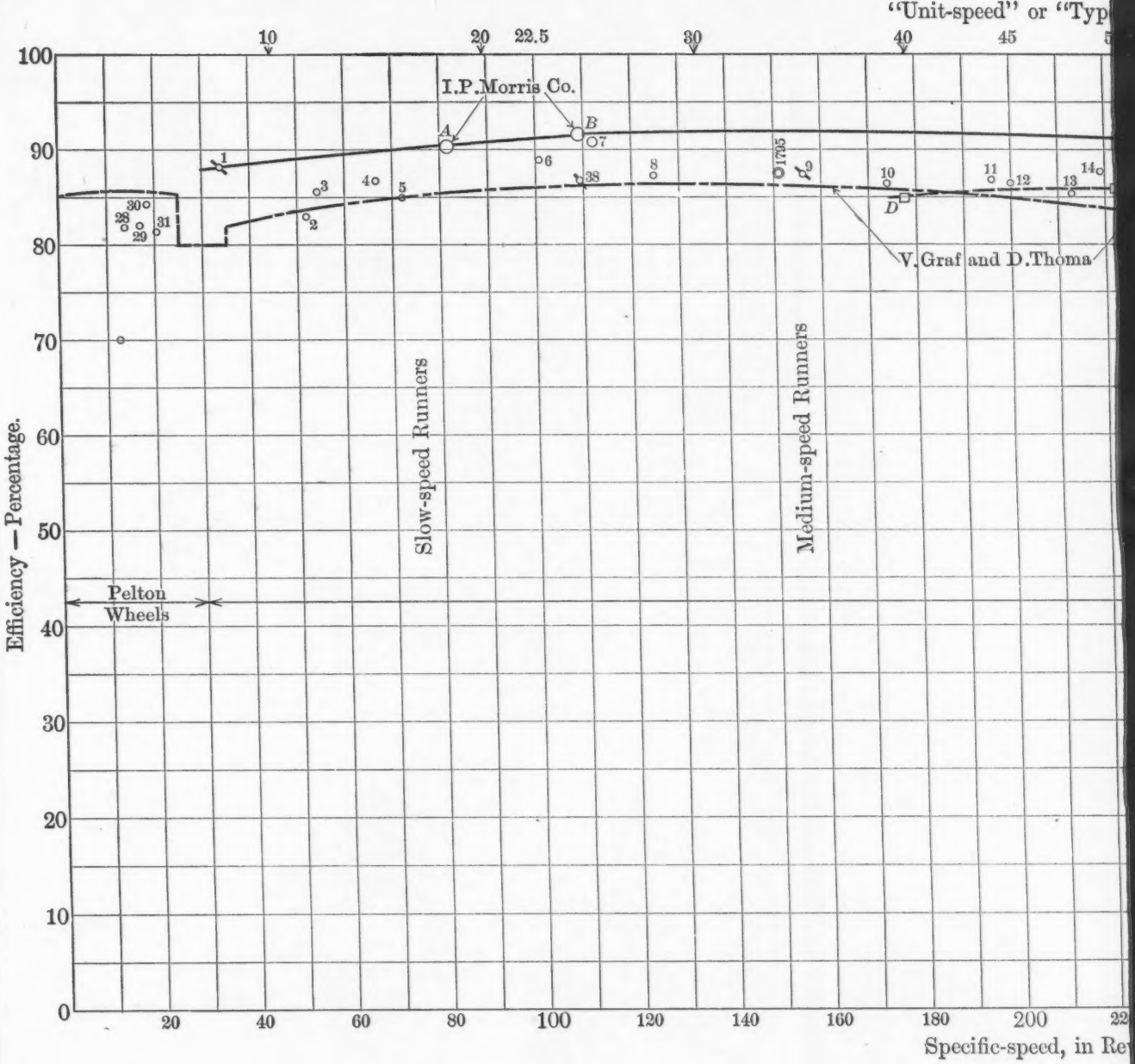
As many of these tests were made at Holyoke, and as the question

* See article by S. J. Zowski, *Engineering News*, Jan. 28th, 1909.

† Published in the *Zeitschrift des Vereines deutscher Ingenieure*, June 29th, 1907.

‡ Recently published in *Engineering News*, September 9th, 1909, p. 286.

Efficiency — Percentage.


"Unit-speed" or "Typ
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45
100
90
80
70
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50
40
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I.P.Morris Co.
A
B
V.Graf and D.Thoma
D
Slow-speed Runners
Medium-speed Runners
Pelton Wheels
Efficiency — Percentage.
20
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60
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100
120
140
160
180
200
Specific-speed, in Re

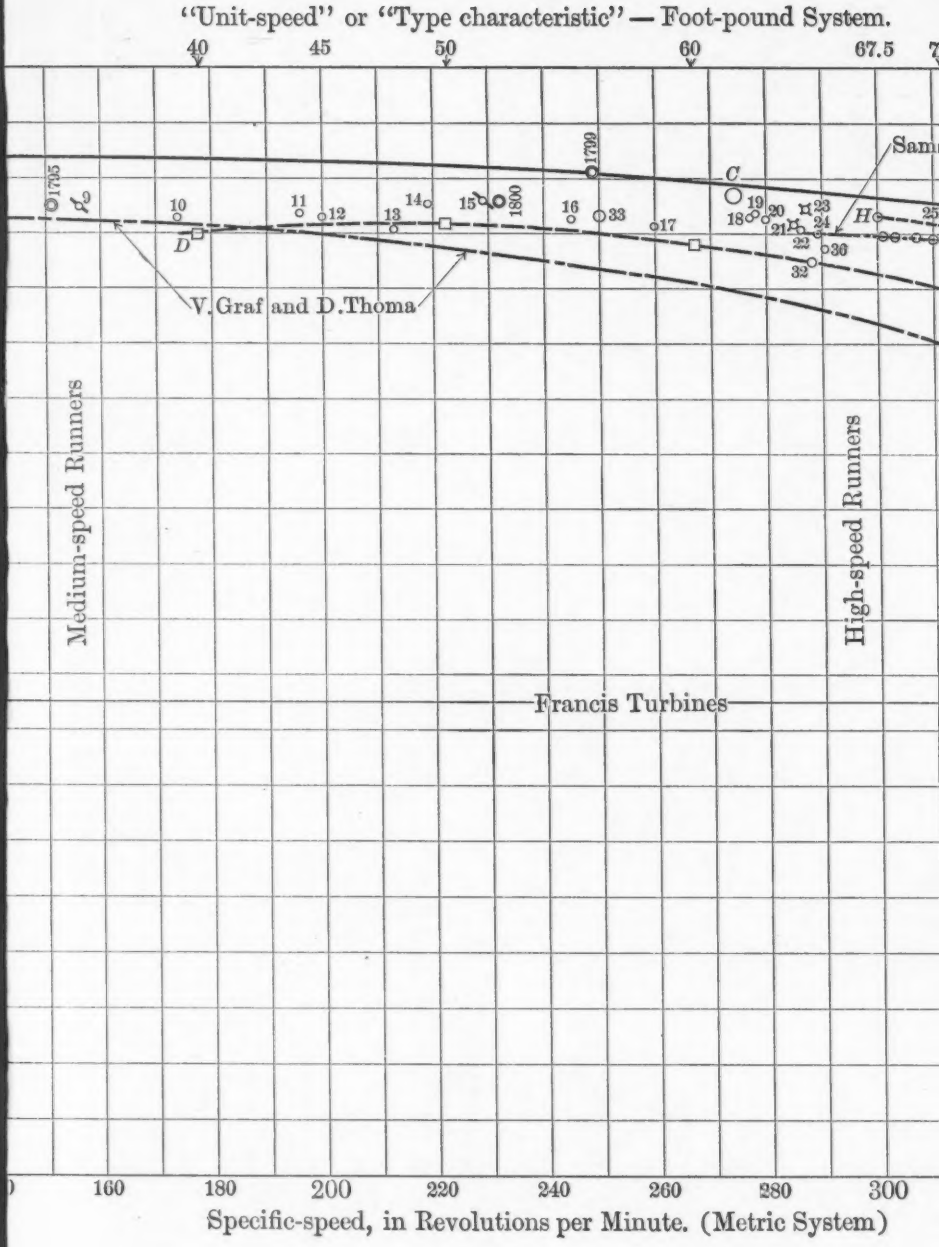
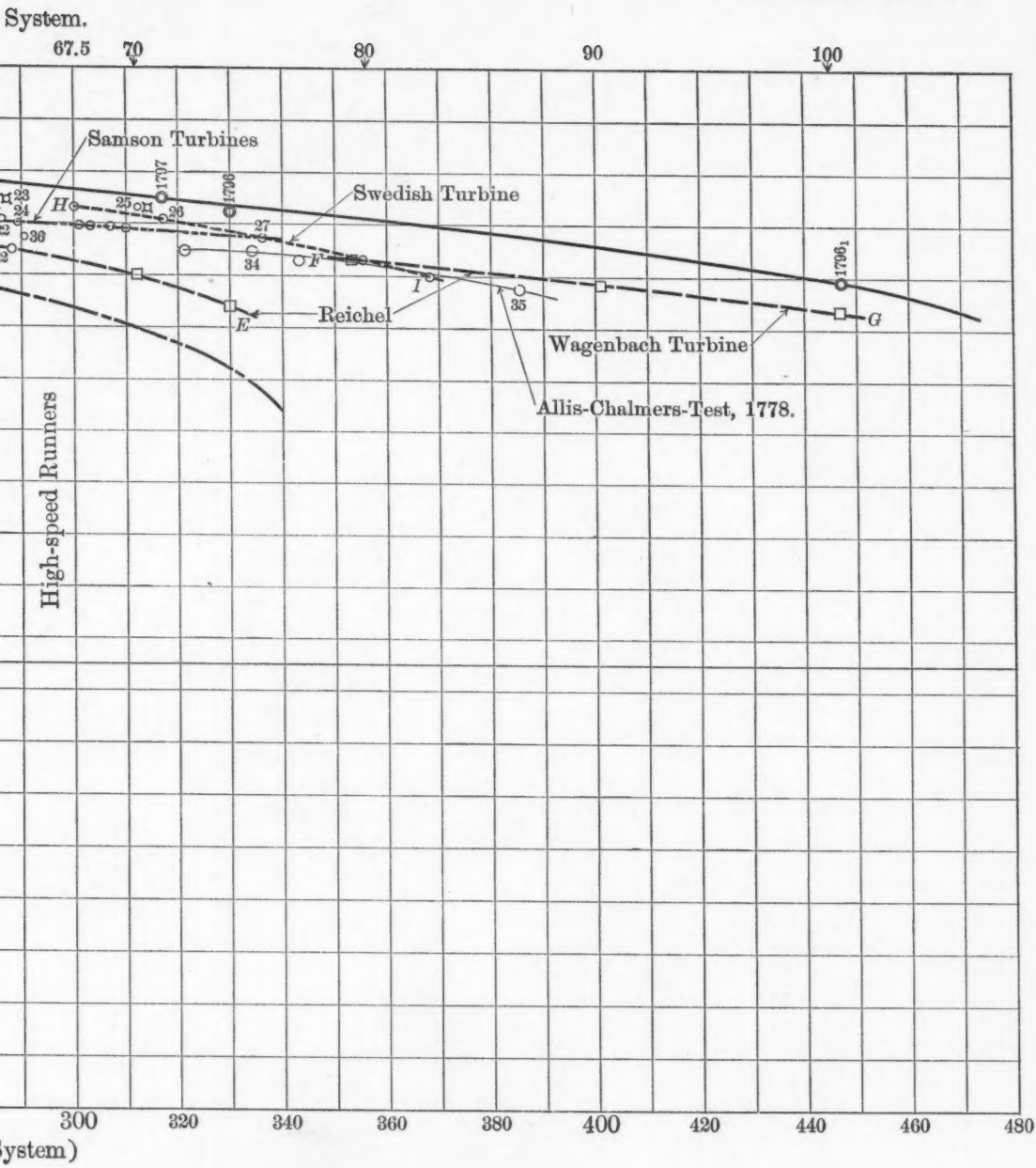

"Unit-speed" or "Type characteristic" — Foot-pound System.
40
45
50
60
67.5
1795
1799
1800
A
B
C
D
H
10
11
12
13
14
15
16
17
18
19
20
21
22
23
24
25
30
32
33
V. Graf and D. Thoma
Medium-speed Runners
High-speed Runners
Francis Turbines
160
180
200
220
240
260
280
300
Specific-speed, in Revolutions per Minute. (Metric System)

PLATE CXIX.
PAPERS, AM. SOC. C. E.
NOVEMBER, 1909.
MODERN HYDRAULIC TURBINES.



of the accuracy of Holyoke tests has already been raised, the writer takes this occasion to state that he has had the opportunity to investigate fully the methods used at the Holyoke flume, and is convinced of the substantial accuracy of the Holyoke results. During the test of a 33-in. turbine built by the I. P. Morris Company, at which the writer was present as the representative of that company, he was able to check up the constants of the flume, measuring the dimensions of the brake, etc., checking the elevation of the weir crest with a level, and checking the head on the weir with an independent glass gauge during the test. Mr. Moody.

No sources of systematic error were found; and the results are believed to be as accurate as the Francis weir formula, within the limits of its use at Holyoke.

The specific speed-efficiency curve, as plotted, passes through the following points:

Point 1 is from a test of a 2 000-h.p. horizontal, spiral-casing turbine with double runner, operating under a head of 122 m., at Rauris-Kitzloch (Salzburg), Austria. It was built by Escher Wyss and Company, of Zürich, for the Aluminium-Industrie A.-G. Neuhausen, and developed a maximum efficiency of 88.2% with a specific speed of 34. The data for this and other tests of Escher Wyss wheels are furnished by the bulletin published by the Escher Wyss Company (1908).

Points *A* and *B* are from tests of two wheels built by the I. P. Morris Company, of Philadelphia, the guides and runners of which were designed by the writer, under the direction of Mr. W. M. White, Hydraulic Engineer of the company. Point A represents the 750-h.p. turbines, built for J. G. White and Company, and installed at the Comerio Falls Plant, Porto Rico, of the Porto Rico Light and Power Company. These turbines are of the horizontal, volute-casing type, with single runner and single draft-tube. They have movable guide-vanes, or "wicket-gates," with outside operating mechanism; and the runners are of bronze. When tested in place, the output being measured electrically and the water by a Francis weir, they showed an efficiency of 90.3% when developing 755 h.p. under a head of 182.75 ft. The speed was 450 rev. per min.

Point *B* is from the acceptance test of unit No. 12 at power-house No. 3 of the Niagara Falls Hydraulic Power and Manufacturing Company. The test was made in place, under operating conditions, by the engineers of that company and of the I. P. Morris Company, and was very carefully conducted. The water measurement was by weir, using the Francis formula. The turbines are of 10 000 h.p., of the horizontal, volute-casing type with double runners and two draft-tubes per unit. They have movable guide-vanes and bronze runners. The test showed that when delivering 8 934 h.p. under a head of 214.3 ft., at a speed of 299.9 rev. per min., the efficiency was 91.7 per cent. Mr. White states in a letter:

Mr. Moody. "I have no doubt but that the 10 000-h.p. units are the most efficient units in operation to-day."

Point 7, just below the curve, is from a test furnished by the Pelton Water Wheel Company on a turbine built by that company for the Black Hills Traction Company at Spearfish, S. Dak. The test showed a maximum efficiency of 90.8%, as read from the curve. The wheel is of the "Pelton-Francis" type, with volute casing and single runner, and was tested in place by the engineers of the companies.

Point *C* is from the Holyoke test of the 33-in., I. P. Morris Company wheel previously mentioned. This wheel showed an efficiency of 88.49% with 17.51-ft. head, 184.57 rev. per min., and 142.18 h.p., the corresponding specific speed being 274.

Other points on the curve are from Mr. Larner's paper, and are indicated by the numbers used by him.

TABLE 9.

Number of point.	Turbine.	Builder.	Place of test.	Efficiency. Percentage.	Specific speed, n_s.
20	Hercules 36-in.	Holyoke Machine Co.	Holyoke Flume.	86.17	280
21	" 36-in.	" " "	" "	86.51	285
23	" 30-in.	" " "	" "	87.07	287
25	" 30-in.	" " "	" "	86.50	314
24	Samson 56-in.	James Leffel & Co.	" "	85.01	289
27				83.58	336
36	" 50-in.	" " "	" "	83.59	291
..	" 45-in.	" " "	" "	84.55	310
..	" 40-in.	" " "	" "	84.72	307
..	" 35-in.	" " "	" "	84.78	303
5	Pelton-Francis	Pelton Water Wheel Co.	In place, Sultepec.	85.0	72.5
10	" "	" " "	In place, Schaghticoke	86.5	174
33	Type E-30-in.	Allis-Chalmers Co.	Holyoke	86.51	250
..	Type F-30-in.	" " "	"	82.31	321
34				82.5	334
..				81.5	343
35				78.89	385
8	10 500-h. p. Shawinigan.	I. P. Morris Co.	In place	87.2	125
3	Samson Niagara Type.	James Leffel & Co.	" "	85.6	54.5
4	6 100-h. p. at Hamilton Cat. Co.	J. M. Voith, Heidenheim.	" "	86.8	66.5*
9	At Rheinfelden	Escher Wyss & Co.	" "	87.4	156.5
15	At Manegg-Zürcher Papierfabrik.	" " "	" "	88.0	229
38	For A. Giraud & Cie., Lyon.	" " "	" "	86.8	109.4
32	Experimental at Sagan.	Designed by Dr. Camerer.	In Germany, Experimental Flume	82.3	288
13	Swain	Swain Turbine & Mfg. Co.	Holyoke	85.39	213†
17	McCormick 39-in.	S. Morgan Smith Co.	"	85.78	260†
19	McCormick 51-in.	J. & W. Jolly.	"	86.84	278†
18	McCormick 33-in.	S. Morgan Smith Co.	"	86.3	277†

NOTE.—Point 11, at 196 specific speed, is Professor Thurston's test previously referred to.
* From "Hydro-electric Developments and Engineering." by F. Koester.
† " "Water Power Engineering," By D. W. Mead, M. Am. Soc. C. E.

Mr. Moody.

The only other turbine known to the writer which, in efficiency, approaches very closely the values given on the curve, is a French wheel built by the *Société des Établissements Singrun,* at Epinal, for the French Government, the efficiency of which is reported to be 90.4 per cent. The data for calculating the specific speed have not been ascertained.

Many excellent results not falling on the curve, but showing very creditable efficiencies and speeds, are shown in Table 9.

In conclusion, the writer wishes again to express his appreciation of Mr. Larner's valuable paper; and to extend thanks, for their courtesy in furnishing information, to the following: Mr. H. Birchard Taylor, Assistant Hydraulic Engineer, I. P. Morris Company; Mr. J. E. Strothman, Assistant Manager, Pumping Engine and Hydraulic Turbine Department, Allis-Chalmers Company; The Holyoke Machine Company, Mr. W. W. White, Manager; The Pelton Water Wheel Company, Mr. J. V. Kunze, Manager; James Leffel and Company; and The Platt Iron Works Company, Mr. John Sturgess, Western Representative. It is regretted that lack of time has prevented the writer from awaiting the data which are being sent by the Platt Iron Works Company.

Mr. Parker.

JOHN C. PARKER, ASSOC. M. AM. SOC. C. E.—In this paper Mr. Larner has contributed so much that is of signal technical interest that one may lose sight of some of his general remarks. The speaker would like to call attention to a very significant paragraph on page 761,* in which the author deals with the advisability of sacrificing, in some cases, the fractional-load efficiency of wheels for the sake of the economical advantage in initial cost of installation. This is of some interest, but the speaker thinks that the general principle laid down should be viewed with caution. As the statement stands, it is undeniably correct, as applied to developments such as those at Niagara Falls, where the maximum quantity of water that may be diverted is limited by law; and also where the quantity of water used at fractional loads is of no importance, being well below that which may be diverted.

It also applies well to those cases where the development is not greater than the minimum flow of a stream. In most installations, however, it will be found economical to over-develop the minimum flow, and to supply a steam reserve to take care of the periods of deficiency in the natural flow. In such cases the steam reserve will stand idle for the major part of the year. It becomes desirable, therefore, to minimize the fixed cost by installing a cheap steam plant (necessarily an inefficient steam plant). The wheels would operate at times under reduced flow, and, unless there is a multiplicity of units, enabling all the units in service at any time to be operated nearly

* *Proceedings*, Am. Soc. C. E., for August, 1909.

Mr. Parker. at their full load, they will be worked uneconomically because the plant is an uneconomical low-load plant. A considerable quantity of coal will be burned, under these conditions. Therefore, in a plant where the minimum flow is less than that required to carry the minimum load which may be thrown on the plant, it becomes desirable to look to the fractional-load efficiencies of the wheels.

Another comment by Mr. Larner, to which the speaker would like to call attention, refers to tests. The data contributed by the author are of so much interest and value that it would be a pity to overlook the facility with which tests on units connected to electrical generators may be made at various speeds, especially of wheels which are not amenable to such tests as may be made at the Holyoke flume.

There is not much mystery connected with the losses in electrical generators. Losses in the power going to waste in the conductors of the armatures of the machines are readily calculated by the application of Ohm's law to readings taken during the operation. The friction and windage in the machines are a purely mechanical proposition, and can be determined by tests in the field. The only other losses are those in the iron in the generators. These losses are a very simple speed function, and can also be readily determined. As the total losses will not be more than from 2 to 4%, even in the larger generators, a slight inaccuracy in determining their variations with the speed is not very serious.

The readiest method known to the speaker for determining these losses as a speed function is by using what is known as a deceleration curve. With the water shut off from the turbine, the machine is brought up to speed by some outside source—possibly electrical—by interconnecting the generators on two wheels, one of which can take water; bringing them up to more than their running speed, and then cutting loose that wheel in which the electrical generator losses are to be determined.

It is then possible to determine losses by the electrical meters and by observing the speed of the wheel as a time function. Knowing the moment of inertia of the rotating parts—which can be readily calculated—observing the negative time differential of the speed, and multiplying by the moment of inertia, one can readily plot the power losses as a time function.

The moment of inertia can be determined experimentally in one of two ways. In a small machine, the addition of a known and readily determinable weight gives a ready solution. Its influence upon the deceleration curve will give the moment of inertia of the masses of the rotating parts under normal conditions.

Another method which the speaker has intended to try (but has not yet succeeded in doing because of lack of time) consists in using an electrical braking device. The simple connection of the electrical

generator through a known resistance, and the measurement of the current carried by it, will show what the power usefully generated does in the way of braking the machine. Determining this moment of inertia and taking the whole group of curves should not take more than half a day, and therefore it would be easy to obtain the electrical losses as a speed function, and thereby extend to large units the very valuable study that Mr. Larner has made with such units as are capable of being tested in the Holyoke flume. The speaker believes that engineers who have facilities should look into the matter and see if Mr. Larner's data cannot be extended to such wheels. Mr. Parker.

Another of Mr. Larner's facts is of great interest, and the speaker is very glad that he has put himself on record as being opposed to the radical application of "standardization," a shibboleth which has been one of the most baneful things in practice, namely, that of taking "goods off the shelf"—the selling of stock goods for any and every purpose.

In building up American enterprises, standardization has been very helpful; but the speaker thinks that those who have had anything to do with the purchase of apparatus and who have attempted its intelligent application, have met, in all lines of manufacturing, with the unfortunate use of "standards" to prevent progress.

The attitude taken by Mr. Larner—who is connected with the manufacturing industries—is certainly most hopeful, and gives promise of some advance in American practice toward standards which heretofore have prevailed almost exclusively in Europe.

Vol. XXXV. NOVEMBER, 1909. No. 9.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

THE REINFORCED CONCRETE WHARF OF THE UNITED FRUIT COMPANY AT BOCAS DEL TORO, PANAMA.

Discussion.*

By Messrs. John Hawkesworth and John B. Cameron.

Mr. Hawkesworth.

John Hawkesworth, Assoc. M. Am. Soc. C. E. (by letter).—The writer has examined this paper with much interest. From the author's statements it is quite clear that, under the conditions, unprotected timber piles could not be used. The scheme of casting protective shells of concrete appears to have afforded a practical solution of the problem. However, it would be interesting to know whether any investigations were made as to the comparative cost and practicability of regular reinforced concrete piles, the greater cost, of course, being offset in part by the smaller number required.

If the concrete shells should develop fine cracks, from temperature stresses, wave abrasion, or the impact of vessels against the wharf, what opportunity would be afforded to the teredo? If the hole made by the teredo is only $\frac{1}{30}$ in. in diameter, as stated, it would not take much of a crack to facilitate his entrance to the wood inside.

In regard to the reinforced concrete design, it appears that the slab reinforcements run between beams and parallel to the 8 by 30-in. girders, the span varying from about $3\frac{1}{2}$ to 10 ft. Although it is stated that the slab was designed for 250 lb. per sq. ft., the amount of reinforcement appears to be just as great for the short as for the long slab spans, and the entire slab load appears to be carried by the beams, which are primarily designed as stringers to carry the tracks. As the span between girders is also never more than 10 ft., it would seem

*This discussion (of the paper by T. Howard Barnes, M. Am. Soc. C. E., printed in *Proceedings* for September, 1909, and presented at the meeting of November 3d, 1909), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

that a more rigid and economical design might have been obtained by running the slab reinforcing bars in both directions, and increasing the spacing of the bars. Mr. Hawkesworth.

The author makes no statement as to the distance allowed between reinforcing bars, and the under surface of beams, girders, or slabs, nor as to the assumption in regard to the width of slab considered as the flange of the T-beams for girders and beams. This renders it difficult to check the strength of the design or its suitability for 24-ton locomotives, but the statement that no shearing reinforcements were needed, the bars being simply cantilevered over the points of support, would seem to be open to criticism.

JOHN B. CAMERON, Assoc. M. Am. Soc. C. E. (by letter).—The writer has read this paper with especial interest as it brings to mind the ravages of the teredo on wooden dock construction in Alaskan waters. Mr. Cameron.

In 1903-1904 the Alaska Central Railway built a dock, using native spruce piling, and although it seemed to be free from the teredo for some time, it was completely destroyed within 18 months.

It having been stated that cottonwood piling would not be infested by the teredo, considerable quantities of this wood were used when the dock was re-driven. The teredo attacked the cottonwood, however, but not until long after the spruce was destroyed. Evidently, the teredo will not enter cottonwood through the bark, and only attacks it at places where the bark has been removed. The bark, being tough, did not peel much under the hammer, and resisted decay for some time after driving.

The opposite was true of spruce, which was almost completely barked by the operations of logging and driving.

The writer cannot state positively that the teredo will not enter cottonwood through the bark, but careful investigation of infested piles led to that conclusion, which the writer believes to be a fact.

This dock was destroyed within 6 months after the second driving, the spruce piling being cut off and dropping out. It was re-driven with creosoted fir from Puget Sound. At the time, several plans for more permanent construction were considered, but the first cost was much in favor of creosoted fir.

The form of construction described by Mr. Barnes would be favorable for docks in Alaska, as local supplies of piling and gravel can be found there.

MEMOIRS OF DECEASED MEMBERS

NOTE.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

HENRY WALLER BRINCKERHOFF, M. Am. Soc. C. E.*

DIED SEPTEMBER 7TH, 1909.

Henry Waller Brinckerhoff was born on May 22d, 1845, in Sing Sing, now Ossining, N. Y., and was the eldest son of Isaac Brinckerhoff, Medical Director, United States Navy, and Mary Gordon (Waller) Brinckerhoff.

His ancestry, on his father's side, was Dutch, and on his mother's, English and Scotch. The Brinckerhoffs were among the earliest settlers of the United States, coming from Holland in 1638, and many descendants of the name in this and neighboring States represent the vigor of this stock, members of which took part in the Revolutionary and subsequent wars.

The Scottish Gordons have been conspicuous in history, and one of that name, General James Gordon, Mr. Brinckerhoff's great-grandfather, was present at the Battle of Bennington. Mr. Brinckerhoff's father served in the Civil War, and he himself, while yet a student, served three months with the 12th Unattached Company of Massachusetts Volunteer Militia.

Mr. Brinckerhoff's early years were spent in Cambridge, Mass., where he obtained his education at the Latin School, Harvard College, and the Lawrence Scientific School of Harvard University. His first work was as an Assistant in the Civil Engineer's office at the United States Navy Yard, Brooklyn, N. Y. In 1869, he was appointed a Civil Engineer in the Navy, and was stationed at the same yard, where he served for some time, constructing quay walls and other naval work. Upon leaving that post, on January 1st, 1870, he was appointed Principal Assistant Engineer on the Jersey City Water-Works, under the late Mr. John F. Ward, in which capacity, he designed the pumping machinery and made the plans for the high-service and new reservoir, also supervising sewer construction and other municipal work.

With the late Mr. Samuel E. McElroy, Mr. Brinckerhoff was engaged upon the Kings County town survey, and afterward made a hydrographic survey of the Passaic River, from Newark to Rutherford Park, for the late General John Newton, Chief of Engineers, U. S. Army, Hon. M. Am. Soc. C. E. In 1874 he was engaged with Messrs. Davidson and Mars, Contractors for the heating arrangements and elevators in the New York City Post-Office Building, his work being

* Memoir prepared by Charles H. Myers, M. Am. Soc. C. E.

the design and erection of the telescopic elevators for that building. Mr. Brinckerhoff remained with this firm for some years, and was afterward engaged for a short time in steam-heating work for Mr. Wyllys H. Warner.

In the autumn of 1877 he took charge of the construction of the foundations of the Gilbert Elevated Railroad from Grand Street to Morris Street, New York City, under the Chief Engineer, the late Mr. William F. Shunk. He performed other work in connection with that structure, and in the following year he inspected the shop work for the Ninth Avenue "Joint Structure" Elevated Railroad. In June, 1879, under the direction of Major George W. McNulty, M. Am. Soc. C. E., Mr. Brinckerhoff designed and inspected the construction of the iron street bridges in the Brooklyn approach of the East River Bridge—the first bridge across the East River, now officially known as "The Brooklyn Bridge."

In 1881, Mr. Brinckerhoff resigned his position on this work, to take charge of the building work of the New York Steam Company, under the late Charles E. Emery, M. Am. Soc. C. E., Engineer and Superintendent for said company. He designed and superintended the erection of the large boiler station on Greenwich Street, New York City, and, after this work was finished, he assisted the late James B. Eads, M. Am. Soc. C. E., in his plans for the proposed ship railway across the Isthmus of Tehuantepec, Mexico. For some time afterward, Mr. Brinckerhoff was Managing Editor of *The Engineering Record,* and, in 1889, made the earliest report on the failure of the South Fork Dam which caused the destruction of Johnstown, Pa.

When the plans for the Broadway Cable Railroad were to be made, Mr. Brinckerhoff was called by Major McNulty, Chief Engineer, to take charge of the office work, and he remained in that position until the road was completed and put into operation. He then began his practice as a Consulting Engineer, being thus engaged until illness compelled a cessation of his active labors shortly prior to his end. His death was caused by an arterial affection which assumed a serious phase in the last few months of his life.

A characteristic of Mr. Brinckerhoff's work was a thoroughness which was noticeable in all his undertakings. As Consulting Engineer, his advice was sought in regard to the foundations of the Johnston Building, at Broad Street and Exchange Place, and other heavy buildings in New York City, besides various other works in the vicinity. Mr. Brinckerhoff had professional experience in both civil and mechanical engineering.

He was most patient and considerate in his dealings with all who came in contact with him, while his honesty and candidness made him outspoken whenever occasion required. His, too, was a kindly and sympathetic nature, with deeply religious sentiment. For a long time

he was a Deacon of the Central Congregational Church in Brooklyn, N. Y., and he took much interest in all missionary work. To the stranger from distant foreign countries, he gave a welcome, often with timely assistance. His church will feel his loss greatly.

Mr. Brinckerhoff was President of the Bedford Dispensary and Hospital, of Brooklyn, N. Y.; Vice-President of the Congregational Club, of Brooklyn; a Member of the Holland Society, and of the Masonic Order. He married, at Cambridge, Mass., on December 25th, 1866, Edith Adelaide Barry, who died on April 19th, 1897. Her loss, and that of his eldest daughter, the wife of the Reverend Winthrop B. Greene, three years later, were heavy blows, which left their lasting effect upon him. A son, Henry Gordon Brinckerhoff, and a daughter, Mrs. Herbert G. Hanford, survive him.

Mr. Brinckerhoff was a loyal friend and a genial companion, and those who asked his advice received cheerful hearing and sincere counsel. Naturally, he took great interest in the early history of his native State, especially in the period of the Dutch occupation. He will be remembered among his acquaintances for the possession of a fund of humor and store of varied information. In his closing days, his patience and serenity never forsook him, and his passing away was peaceful and painless.

Mr. Brinckerhoff was elected a Member of the American Society of Civil Engineers, on November 7th, 1883.

HORATIO SEYMOUR, Jr., M. Am. Soc. C. E.*

DIED FEBRUARY 21ST, 1907.

Horatio Seymour, Jr., was born in Utica, N. Y., on January 8th, 1844. He was the son of John F. and Frances A. (Tappan) Seymour, and the nephew of Horatio Seymour, a former Governor of the State. He was educated in the Academy at Utica, and at Yale College, from which he was graduated in 1867. In 1869-70 he attended Yale Scientific School.

His first engineering service was as an Assistant on the Cazenovia and Canastota Railroad. Afterward, he was with the Seneca Falls and Sodus Bay Railroad, the Wellsville and Lawrenceville Railroad, and was Chief Engineer of the Cowanesque Valley Railroad. In 1873 and 1874, he was with the Fall Brook Coal Company and the Buffalo Coal Company. From October 1st, 1874, to December 31st, 1875, he was an Assistant Engineer in the State works of New York on the Erie Canal, and was afterward engaged on the topographical survey of the State. He became State Engineer and Surveyor of the State of New York, on January 1st, 1878, when he was 34 years old, and served two terms of two years. He was selected for that office as one of the representatives of the spirit of vigorous reorganization of the management of the great public works of that State, which had been inaugurated three years before by a Governor who notably elevated the standard of public service, and whose work in this reform had been ably enforced by another Member of this Society, the immediate predecessor of Mr. Seymour.

Mr. Seymour then went to Marquette, Mich., where he engaged in general practice, but soon took charge of the interests of the Michigan Iron and Land Company, which owned large land properties, including iron mines and extensive tracts of fine timber. The management of these properties occupied him fully until 1903, when he returned to Utica, where he resided until his death.

Mr. Seymour's notable professional service was in connection with the State works of New York. While an Assistant Engineer, his declination to certify certain estimates without examination, and his protests against the unlawful demands of contractors, were, in part, the basis of an extended investigation of canal frauds and a radical reform in methods.

As State Engineer, Mr. Seymour conducted the department with ability and economy. His four reports are notable State papers. The first, written after a year's incumbency of the office, discusses the value of canals and water routes for internal commerce, their relation to

* Memoir prepared by John Bogart, M. Am. Soc. C. E., and Alfred Noble, Past-President, Am. Soc. C. E.

railroads, and their influence in maintaining reasonably low transportation rates. It calls attention to the danger that traffic might be diverted to the water route through the Valley of the St. Lawrence, where the canals are being enlarged and deepened, and argues that a method must be found to lessen the cost of carrying freight by the Erie Canal route, and thus save the commerce to the State and City of New York. This could be done, either by increasing the tonnage of the boats or by increasing their speed. The report advises that the depth of water in the canal should be increased 1 ft. This would add to the speed of the boats and enable them to make an additional trip during the navigation season, or it would add largely to their carrying capacity. The use of power to operate the locks is advocated as an additional means for saving time in transit. The report shows that the Erie Canal had paid into the Treasury of the State about $42 000 000 more than its cost, including all expenditures for repairs and management.

In his second yearly report, Mr. Seymour repeats and enforces the recommendations of the first. In his third report he shows that the market of New York City gives value to property throughout the State, that that city pays about one-half the taxes of the State, and that commerce must be saved to the city and the State. The canals furnish a cheaper means of transportation than any other method. By making the cost of this still lower, commerce will be saved. The plan of adding 1 ft. to the depth of water commends itself to the boatmen, because it requires no outlay on their part, and this increase in depth can be secured by the expenditure of about $1 000 000. Mr. Seymour's fourth and last report advocates the abolition of tolls on the State canals.

The plan of adding to the capacity of the canals by an increase of 1 ft. in the depth of water was seriously considered during subsequent years, and was always known as the Seymour plan. It was not carried out, but the information gathered by Mr. Seymour, and the suggestions made in his reports, were, in a large way, elements in the extended studies which finally resulted in the adoption of the barge canal project, now in active construction.

Mr. Seymour's service as State Engineer was marked by professional ability and excellent business management. These qualities characterized also his management of the large interests subsequently under his care in Michigan.

In 1880 Mr. Seymour married Miss Abigail Adams Johnson, who, with a son, Horatio, and a daughter, Mrs. Henry St. Arnauld, survives him. He was a member of the Sons of the Revolution and of the Society of the Cincinnati.

Mr. Seymour was elected an Associate of the American Society of Civil Engineers on January 8th, 1873, and a Member on April 7th, 1880.

CARL WILLIAM BIRCH-NORD, Assoc. M. Am. Soc. E.*

DIED SEPTEMBER 15TH, 1909.

Carl William Birch-Nord was born in Christiania, Norway, on October 7th, 1880. He was graduated from the Christiania Elementary Technical School in 1899, and afterward completed a post-graduate course in Electrical Engineering at the Institute of Technology at Bingen, Germany.

In 1902 Mr. Birch-Nord came to the United States and settled in Chicago, Ill., where he held positions as Draftsman with Burnham and Company, the Metropolitan Elevated Railway Company, etc. In June, 1903, he entered the service of the American Bridge Company, as Estimator and, later, as Assistant Engineer, in the Estimating Department of the Chicago Plant, where he remained until his death on September 15th, 1909.

On May 15th, 1907, Mr. Birch-Nord was married to Miss Marie Halvorsen, who survives him.

Mr. Birch-Nord was elected a Junior of the American Society of Civil Engineers on October 30th, 1906, and an Associate Member on November 4th, 1908. He recently contributed a paper on "The Design of Elevated Tanks and Stand-Pipes" to the *Transactions* of the Society. He was also a Member of the Western Society of Engineers, and of the Technical League.

* Memoir prepared by Albert Reichmann, M. Am. Soc. C. E., and J. H. Hoff, Esq.

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AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

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AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

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UNDERPINNING THE CAMBRIDGE BUILDING, NEW YORK CITY.

By T. Kennard Thomson, M. Am. Soc. C. E.

To be Presented February 2d, 1910.

In July, 1907, the writer was retained by Henry G. Opdycke, Assoc. M. Am. Soc. C. E., to report on the undermining of the Cambridge Building, on the southwest corner of Thirty-third Street and Fifth Avenue, New York City, by the Pennsylvania Railroad Tunnel, then under construction.

Mr. Opdycke had already made a careful survey of the premises, and also had a number of excellent photographs, from which those illustrating this paper have been selected.

Fig. 1 shows the approximate cross-section of the tunnel excavation at a point opposite this building, as taken by Mr. Opdycke and the writer. The space marked "To be back-filled" represents the approximate deviation from the theoretical tunnel lines, and shows that at one point an apparently misdirected blast caused a cavity extending some 3 ft. south of the building line of the Cambridge Building, or 9 ft. farther south than intended.

It also indicates roughly where Thirty-third Street caved in, on December 14th, 1906, in front of the Waldorf. The cave-in had been

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

back-filled, but the curb, the street, and the Cambridge Building had continued to settle.

Fig. 1, Plate CXX, shows very plainly the settlement in Thirty-third Street. The street had been flooded, and the water covered the curb, in front of the entrance and near the middle of the Cambridge Building, which was the point where the greatest settlement occurred. The tendency of the whole building, as shown by all the photographs, was to bulge toward the north, as well as to settle vertically.

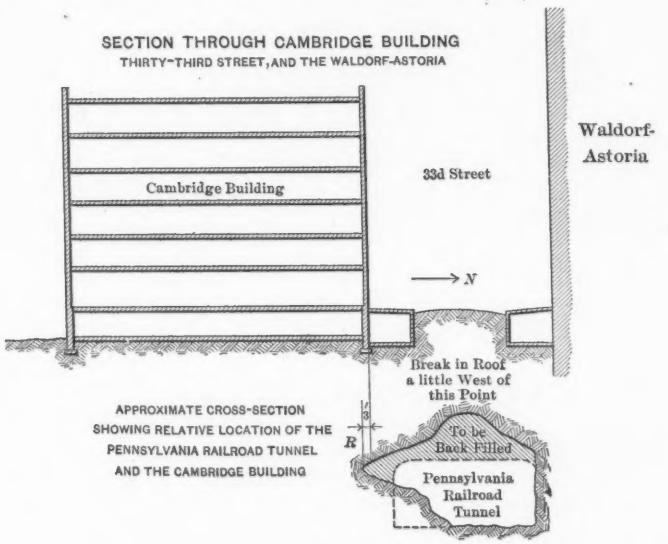


FIG. 1.

It was claimed by others that the whole trouble was caused by a defective 8-in. column just east of the entrance, but it is peculiar that, although this column had moved northward and had settled, it did not break its bed nor its connection with the brickwork of the north wall, and remained absolutely plumb.

There were a number of vertical and inclined cracks in the interior north and south walls, showing the tendency of the building to move northward and at the same time to settle vertically, the dimensions of the cracks decreasing as the distance from the curb increased; in fact, the greatest disturbance in the building itself was at the center of the north wall, and in the areaway, sidewalk, and street, as might be expected.

The connection between the two boilers in the cellar parted, break-

ing the pipe connections. Both boilers settled, the one several inches farther north than the other.

There were several cracks in the area floor, and some disturbance of the area wall, one joint alone opening up more than 1 in.

The sidewalk settled considerably, as shown in Fig. 2, Plate CXX; and there were some cracks. The curb was reset several times, sometimes as much as 7 or 8 in.

Fig. 1, Plate CXXI, shows some of the cracks in Kirkpatrick's Jewelry Store, in the Cambridge Building. In a number of places the joints in the marble work in the main hall opened up.

The fact that an old stream ran under the site of the Cambridge Building and the Waldorf-Astoria, also aggravated matters, especially as the break in the tunnel was at or near this old stream. The foundations of the Waldorf-Astoria had been carried down to the rock, but the east end of the Cambridge Building was founded on a spread footing of concrete and the west end on piles.

When the Cambridge Building was remodeled, in 1906, to convert it from a hotel into an office building, these piles, which were about 30 ft. long and, when driven, had been cut off below the water line, were found to be slightly above the water line, the latter having been lowered enough to expose their tops to the atmosphere, allowing them to rot.

It is known, of course, that timber will last practically forever if entirely submerged, but will soon rot out if not entirely under water. Therefore, at that time, the piles were cut off below the new water line, capped with concrete, and five heavy 20-in. I-beams, about 40 ft. long, were placed on the concrete and buried in it, making a very substantial foundation for the west end of the building.

In 1907, when Mr. Opdycke and the writer examined the tunnel, they noticed that several rather large streams of water—evidently from the old stream shown on the maps—were pouring into it at this point. The examinations showed that the building had settled considerably and was still settling, and the writer was requested to design a system of underpinning carried to rock. This he refused to do at that time, for fear of opening up a direct connection between the cylinders and the tunnel, thus endangering the entire building; but he recommended tying back the front wall with rods, to prevent it from falling away from the remainder of the building.

FIG. 1.—SETTLEMENT OF STREET, CURB, ETC., IN THIRTY-THIRD STREET, IN FRONT OF CAMBRIDGE BUILDING.

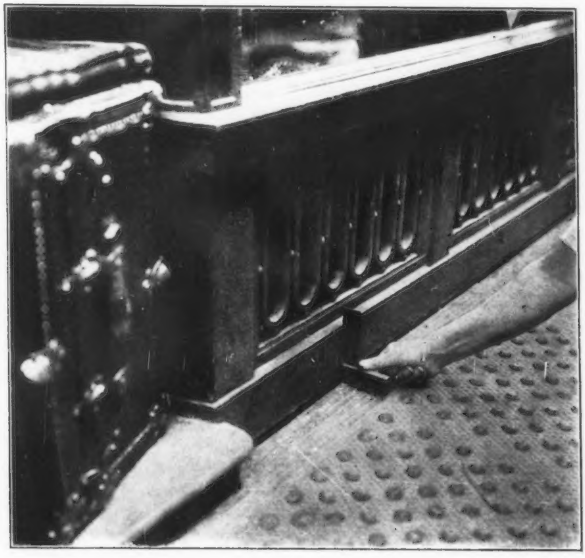


FIG. 2.—SETTLEMENT OF SIDEWALK, IN THIRTY-THIRD STREET, IN FRONT OF CAMBRIDGE BUILDING.

This was immediately done, as shown by Fig. 2. Two 12-in. beams, about 41 ft. long, were placed vertically just east of the main entrance on Thirty-third Street, which was the point of greatest disturbance. These beams extended from the areaway floor up to and slightly above the third floor. The tops of the beams were anchored back by a 1½-in. rod, about 102 ft. long, made up of stock lengths of about 16 ft., and connected with turnbuckles.

The rear or south end of the rods ran through an open court, but close to the wall. On each side of the court 12-in. I-beams, about 6 ft.

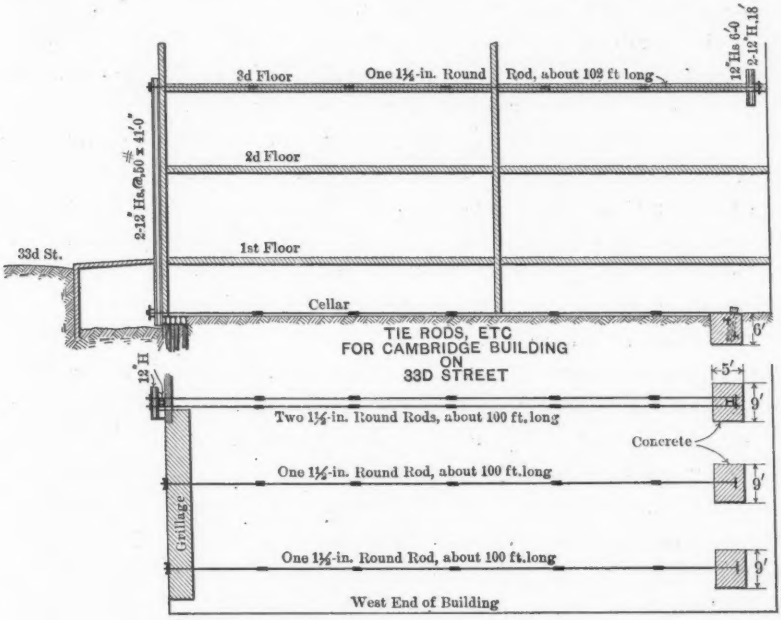


FIG. 2.

long, were set vertically, and formed seats for the two 12-in. beams (18 ft. long) to which the rod was anchored. The north end of the rod was placed just under the floor in a private office, and through the main hall.

The bottom ends of these long beams were anchored with two 1½-in. rods, just above the cellar floor. These rods were about 100 ft. long, and were made up of short sections, the rear or south ends being anchored to 5 by 9-ft. blocks of concrete, about 6 ft. deep, placed just under an old north and south partition wall, and in a position slightly

inclined from the vertical. Two horizontal beams were also placed in the concrete, as shown.

Near the west end of the building, another 1½-in. rod was placed, just above the cellar floor, the north end being hooked over the top of the five 20-in. I-beams, and the south end being anchored to a similar block of concrete.

Half way between this westerly rod and the first pair, a similar rod was placed and anchored in the same manner as the westerly rod.

The 41-ft. beams in front were then encased in sheet metal on account of their unsightly appearance.

The writer's examination was completed about July 23d. A contract was at once made with Mr. Alexander Brown, Jr., to tie back the building, and the work was completed in about a week.

The rods were left in place until the permanent underpinning had been completed (during the following year), and effectually prevented the north wall from pulling away, although the entire building had apparently moved north in the interval. The anchorages in the cellar, although made very heavy, showed signs of the great strain, by the cracks which opened up in the concrete floor behind them.

When the rods were first put in, in the autumn of 1907, the writer tested them to see that they were under sufficient initial strain, and when he was called in to make another report and examination, in the following spring, a similar test indicated that they were under a considerably greater strain, showing the tendency of the building to move north. In the fall of 1908, when these rods were to be removed, the men commenced to saw through the second rod from the west on the cellar floor, quite near the end attached to the 20-in. I-beams, without first slacking up the turnbuckles. When they had sawn three-quarters through the rod, it broke with a snap, and the long end sprang back about 2 ft. The men followed their instructions with the remainder of the rods, and slackened off the turnbuckles first.

The examination of the building, in March, 1908, indicated that there had been considerable vertical movement or settlement since the previous fall. Borings then taken showed that the rock was about 20 ft. below the curb at the east end of the building and about 44 ft. at the west end. The substrata were found to be partly of made ground, and below that there was sand, gravel, clay, and silt or muck, such as found in the beds of all old New York streams—a very treacherous material to have under a building.

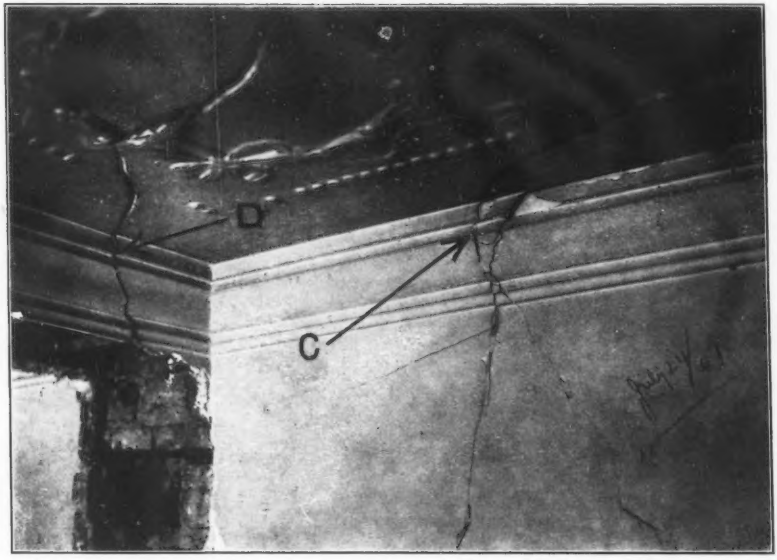


FIG. 1.—CRACKS IN WALL AND CEILING, MAIN FLOOR, CAMBRIDGE BUILDING.

FIG. 2.—SETTLEMENT IN AREA, CAMBRIDGE BUILDING.

The borings also indicated that the old stream had been entirely diverted by the tunnel, leaving the piles dry, and making their ultimate destruction by rot certain.

In the meantime the tunnel excavation had been completed, and the permanent masonry lining built in place, and it was thought that the stream which had been flowing into the tunnel the year before would back up, when it could not get into the tunnel, and again submerge the piles. The borings, however, indicated that this was not so, but that the tunnel had in some manner effectually and permanently diverted the old stream, thus lowering the ground-water level 30 ft. or more.

Reports had been obtained from H. G. Opdycke, Assoc. M. Am. Soc. C. E., Rudolph P. Miller, M. Am. Soc. C. E., Daniel E. Moran, M. Am. Soc. C. E., and Mr. Joseph Caccavajo, and, as these were similar in nature to that made by the writer, the Gorham Company (the lessee), decided to underpin the building to rock, and a contract for doing so was made with Mr. Alexander Brown, Jr., who retained the writer as Consulting Engineer to devise a plan and carry it out.

The east end of the north wall of the Cambridge Building was occupied by the show windows of the Kirkpatrick Jewelry Store, on the Fifth Avenue and Thirty-third Street sides of which store the old brick wall in the first story had been removed and a number of 8-in. cast-iron columns had been inserted to carry the upper brick wall and floors.

This is a kind of construction very frequently resorted to in converting old buildings into stores, etc., and answers the purpose as long as the foundations are not disturbed, or the walls jacked against, as there is no transverse bracing between the columns; therefore it was decided not to use the old system, which involved jacking against the building to force cylinders or walls into the ground.

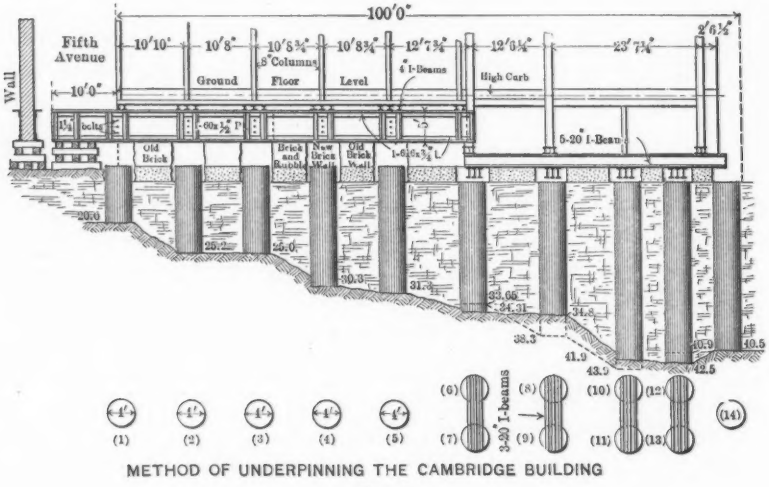
Under this end of the wall, from the curb to the cellar, there was a wall of brick and rubble masonry with some ashlar facing, which had been built, as was customary at the time and for that class of work, without any engineering supervision or advice; in other words, it was of the usual poor construction.

This foundation wall rested on a concrete footing, and on top of the wall were set the 4-in. beams, which for the length of the wall, formed a base for the 8-in. cast-iron columns.

As it was not advisable to do much cutting in this wall in the condition it was then in, it was decided to reinforce it by placing a 5-ft.

plate girder on each side about 1 ft. below the 4-in. beams, to bolt these two girders together with $1\frac{1}{4}$-in. bolts, and to grout the spaces between the girders and the wall. These spaces were usually 1 in. or more in width, as no attempt was made to dress down the surface of the masonry. As these girders in place were to be about 67 ft. long, they were each shipped in two sections in order that they might be put in position readily.

The west end, shown by Fig. 3, rested on the end of the old 20-in. steel beams, and the south end on a timber blocking under the sidewalk of Fifth Avenue.



METHOD OF UNDERPINNING THE CAMBRIDGE BUILDING

FIG. 3.

These girders had $\frac{1}{2}$-in. web plates and one 6 by 6 by $\frac{3}{4}$-in. angle in each flange.

After the underpinning was completed, the 10 ft. under Fifth Avenue was sawn off, and a space of about 4 ft. was cut out for a door just west of the splice, the splice having been placed at that point on account of the door.

The girders were placed a little more than 2 ft. 2 in. apart, back to back, and, with the exception of the south end and the space cut out for the door, were eventually bricked up on both sides, making a solid wall on top of the cylinders. As soon as the cylinders were in place and grouted, three 9-in. beams were placed under the 4-in. beams and under each column. The ends of these 9-in. beams were wedged up on

top of the 5-ft. girders, thus transferring the load from the columns directly to these girders. The holes around these short beams were then filled with concrete and brickwork. There were six 8-in. cast-iron columns resting on this pair of girders, and it was now considered safe to undermine two columns at a time in order to make room for the underpinning cylinders, one of which was put directly under each of the five most easterly columns.

As the sixth column and the remainder of the wall, or the portion from the east side of the entrance to the west end of the building, was being carried on the five 20-in. grillage beams, resting on the concrete-capped piles, it was thought that it would be too expensive and dangerous to try to put the cylinders directly under the columns, especially as it would have been necessary to cut out the old piles all the way down. Therefore it was decided to sink four pairs of cylinders, as close to the grillage beams as possible, and then remove enough concrete to place 20-in. I-beams under the old grillage beams, the ends of these short beams to rest on the pair of cylinders, and wedges to be driven between the short beams and old long grillage beams. Three of these short beams were inserted over each pair of cylinders. At the extreme end an additional cylinder was sunk on the center line of the wall, making, in all, fourteen cylinders for the support of the building.

Two additional cylinders were sunk in the areaway, to support the north end of the boilers and the area wall.

Mr. Brown called for bids on this plan, and awarded the contract to Thomas B. Bryson, Assoc. M. Am. Soc. C. E., as his bid was less than half as great as the other bids, and it was concluded that he could do a good job and make a fair profit, both of which he did. He also gave a bond to the owners for twice the amount of his contract. His total contract, including the extra work, was about $13 000, which included supplying and sinking the sixteen cylinders, filling them with concrete, supplying and placing the girders and beams, doing the necessary wedging, etc., and back-filling with concrete where necessary.

The cylinders (Fig. 4) were 4 ft. in diameter, and had very thin shells. The method of placing them constitutes a new system of underpinning; for, instead of placing the bottom section first and jacking it into the ground, and then placing another section on top and repeating the operation, a space was excavated for the top section, which was made 4 ft. deep, and the space outside of this top section was back-

filled, generally with concrete, and remained permanently in that position.

A laborer then entered the top section and, with a short-handled shovel, excavated the material under the "cutting edge" sufficiently to insert one segment of the next ring. All the rings below the top

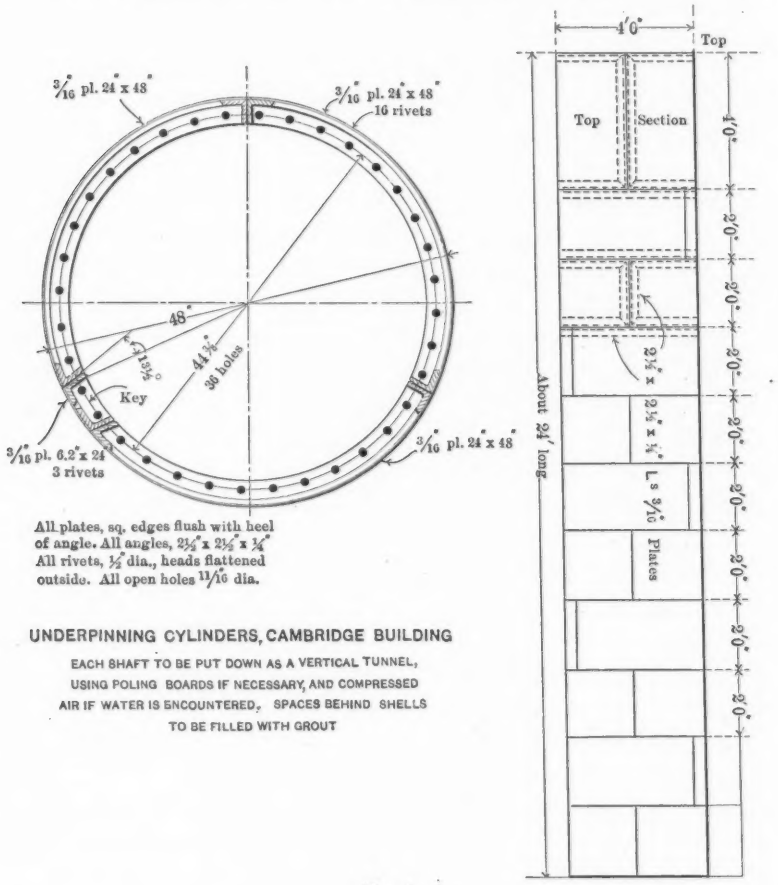


FIG. 4.

section were 2 ft. deep and in four segments, one of which was small, to act as a key, as in tunnel lining. In this key segment the connection angles were bent $13\frac{1}{2}°$ from the radial, to permit putting the key in place. The joints of the other segments, of course, were on the radial lines. All the plates were $\frac{3}{16}$ in. thick, and all the angles (horizontal

and vertical) were 2½ by 2½ by ¼ in. When the first segment under the top section had been bolted in place, the excavation was made for the second segment, then the third, and then the fourth or key. Where the ground was good, the excavation was made for all four segments at once.

Although the borings showed that all the water had been drained off, it was considered desirable to provide in the contract for doing the work under compressed air if water should be encountered, and the first cylinder to be sunk had gaskets in the joints; but, as it was found that the borings had been correct in showing the absence of water, no effort was made to obtain water-tight joints in the other fifteen cylinders.

In sinking one of the cylinders the rock gave evidence of having been blasted, as it undoubtedly had been, as this cylinder was directly over the tunnel. The flushing of the street near the Waldorf allowed considerable water to percolate and get into the uncaulked joints of the cylinder, sometimes in quite a stream, but it was noted with surprise that this water disappeared as soon as it reached the bottom of the cylinder, passing into the drains or crevices alongside of the completed tunnel, although these drains were supposed to have been blocked off or choked up.

This proved the existence of the second danger feared for the Cambridge Building, resulting from the construction of the tunnel. The first was the rotting of the piles due to the removal of the ground-water, and the second the removal of the subsoil, especially where it was muck or silt, as in this case, by the continual draining of the water from the streets, old streams, etc., into or alongside of the tunnels, for it is known, of course, that such continual drainage must carry a large amount of soil or muck with it, causing a gradually enlarged cavity which, sooner or later, would be filled from above, perhaps suddenly, and with disastrous results.

This action is occurring in so many places in New York City that some day we may expect to see a whole building suddenly collapse, like some of the villages in mining districts.

These plans were drawn up in July, 1908; Mr. Bryson was awarded the contract early in August, and by August 21st he had the first cylinder on rock, having spent 4 or 5 days on it, as his men were new to the work, and extra precautions were taken against water, etc. The other

cylinders were put down at the rate of about 2 days each for a depth of about 30 ft. below the curb, the sixteenth or last cylinder being completed exactly 30 days after the first was landed. Generally, an attempt was made to work on two cylinders at a time, but in such cramped spaces it was difficult and subject to delays. Mr. Bryson sublet the placing of the four 33½-ft. girders to Mr. Frank M. Tench, who handled the ticklish problem without disturbing the sidewalk in front of the jewelry store. As the cellar under this store was a salesroom for valuable goods, such as statues, paintings, etc., it was necessary to build a tight wooden partition, about 5 ft. south of the wall which was reinforced by the girders, before work was started, and, after the girders had been bricked in, furred and plastered, the rich red burlap was replaced before the temporary partition was removed.

In order to get permission from the tenants, Mr. Brown had agreed to restore the premises of the Kirkpatrick Company by the end of September, on account of the fall trade, and, by working day and night, he succeeded in doing this.

Table 1 shows the order in which the cylinders were sunk, their depths, etc.

TABLE 1.

Order of sinking.	No. on plan.	Date of reaching rock.	Depth of rock below curb, in feet.
1	11	Aug. 21-08	41.9
2	10	" 26-08	43.9
3	8	" 29-08	38.3
4	9	" 31-08	34.8
5	13	Sept. 5-08	42.5
6	12	" 7-08	40.9
7	4	" 8-08	30.3
8	2	" 9-08	25.2
9	5	" 11-08	31.3
10	3	" 12-08	25.01
11	1	" 14-08	20.0
12	7	" 14-08	34.3
13	6	" 16-08	33.6
14	14	" 17-08	40.5
15	15	" 18-08	41.4
16	16	" 21-08	36.0

It might be stated here that although there was continual settlement up to the time that this building was underpinned, there has been absolutely none since. At one time, when the building was being tied back, the settlement of the area wall broke a 4-in. water main, which caused a good deal of worry, as the water started to flow around the

boilers, and soon would have found its way into the tunnel. It took 3 hours for the proper authorities to cut off this pipe. However, as soon as the pipe broke, the men made a dam of sand bags, thus keeping most of the water away from the low spot around the boilers and allowing it to escape into the sewer.

The writer wishes to thank those connected with this underpinning work, namely, Mr. John S. Holbrook, Vice-President of the Gorham Company; Henry G. Opdycke, Assoc. M. Am. Soc. C. E., Consulting Engineer; Mr. Alex. Brown, Jr., General Contractor; Mr. Frank M. Tench, Erecting Contractor; the Cockburn Barrow and Machine Company, which supplied the steel cylinders, and Lewis F. Shoemaker and Company, which supplied the steel girders, for the way in which each did his share in stopping the settlement of this building.

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PAPERS AND DISCUSSIONS

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BUILDING AGREEMENTS.

By William B. Bamford, Assoc. M. Am. Soc. C. E.

To be Presented February 2d, 1910.

This paper has been limited to agreements for the construction of buildings, and no attempt has been made to cover the subject of agreements in general, because the writer has given this type special study and also feels that the principles underlying such agreements are readily adaptable to those for other construction, and that, within the limits of this paper, many of the vital questions at issue can be brought out more effectively by taking a concrete case rather than by dealing in generalities.

From the practical standpoint of the building agreement, it matters little whether the person who planned the building and will have charge of it until completion is called an Engineer or an Architect. The only practical difference would be that the word "Engineer" would be used in one case and the word "Architect" in the other, to represent such a person. Therefore, it should be understood that wherever the word "Engineer" is used throughout this paper it is intended to designate a person properly equipped by education, experience, training, or temperament, to lay out, supervise, and direct, in an executive capacity, all work, at any stage, in the erection of a building, from the time it is mentally conceived until it is finished and ready for use.

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

The purpose of this paper is not a defense of the Contractor, as against the Engineer or Architect, but is simply one advocating justice and equity to all parties to a contract. Destructive criticism is easy, but constructive criticism is of a different mould, and should always be welcomed among broad-minded men. In suggesting a new form of Agreement and Schedule of Conditions of Contract, it is hoped that something definite may be accomplished in righting the wrongs that most Engineers will readily admit exist, and nearly everyone has tried by different means to correct. The writer does not imagine that he has discovered some new and patentable truth, but hopes that his studies may have brought into concrete shape some practical suggestions for correcting well-recognized faults. After Engineers and Architects have set about putting their own houses in order, as typified by contract requirements, they can then more fearlessly demand that Contractors right the wrongs for which they alone are to blame.

Much might be said on the general subject of fairness and justice to both Contractor and Owner in the various contract requirements, on the faults of omission and commission by Engineers and Architects, and on their duties, obligations, and responsibilities. These subjects would have a direct bearing on the principles underlying the reasons for many of the contract requirements herein suggested. The legal questions involved in contract requirements are also of great importance. In order, however, to keep this paper within reasonable bounds, its subjects have been abridged, but reference has been made to articles and books where much additional information may be obtained, or where some subjects, only touched upon here, will be found in more amplified form.

Historical Summary.

In gathering material for a prospective book on Building Specifications and Contracts, numerous recent specifications and contracts were specially obtained from many prominent architects and engineers for the purpose of study and examination. A careful examination of these, as actual working instruments, and a study of recently published works or addresses, have led to the belief that there was probably a need for investigating and discussing the subject of Agreements (or so-called Contracts) in general, and Building Agreements in particular.

The need for a more or less authoritative set of requirements,

applicable to a proper building agreement, has been recognized for many years. To meet such need, the American Institute of Architects, in co-operation with the National Association of Builders, adopted, a number of years ago, the "Uniform Contract." This contract has remained in force, with no very material changes, up to the present time. The last revision was copyrighted in 1905. This contract, with all its defects, is in general use among Architects, as there is no other form, having the endorsement of a representative American organization, to take its place.

In justice to the American Institute of Architects, it should be said that a Standing Committee on the Standardization of Documents was appointed in February, 1906, and has done much excellent work in collecting material for a new building agreement. Early in 1908 the Committee published, for limited circulation, a pamphlet containing a tentative series of forms, to facilitate the making of suggestions or criticisms to assist the Committee in its further study.

It was recently learned from several authoritative sources that the work of the Committee might not be published for some time, possibly years, and, as the writer's work has been entirely independent of the Committee, he has decided to present its results to this Society for discussion.

Several years ago, the writer learned that the Royal Institute of British Architects had spent many years in perfecting a just and equitable building agreement, but investigation revealed the fact that although the Institute had commenced the work more than thirty years ago, it was almost unknown in America.

The more closely the writer analyzed his own work and ideas and compared them with those of the R. I. B. A., the more firmly convinced he became of the excellence of the latter's work. Looking back at the time which has been spent in perfecting this work, and the number of eminent and experienced men who have contributed to it, there should be small wonder at its present superior condition.

While an agreement for which originality could be claimed for the whole, might be of greater personal benefit, it was decided to sink such personal consideration and frankly undertake to revise a most excellent document, to meet American conditions. The Agreement and the Schedule of Conditions of Contract put forward by the writer have been modified, after serious study, from those adopted by the

R. I. B. A., and the language of the original has been retained wherever possible. Such a revised document has a very important practical advantage over an attempt at originality. As many of the requirements vitally modify traditional American practice in many respects, it might be objected to because it has not had the test of sufficient experience to make it advisable for general use. English and American jurisprudence are fundamentally the same, and therefore the laws or judicial decisions of one country are more nearly similar to those of the other than is true of any other two countries. Since the work of the R. I. B. A. was undertaken originally about 1878, and the present documents were approved in substantially their present shape in 1895, and approved in their present shape in 1903 by the Institute of Builders and the National Federation of Building Trades Employers of Great Britain and Ireland, as well as by the Royal Institute of British Architects, they have had, in Great Britain, the test of time and of litigation, and can therefore be accepted with greater confidence as a secure foundation upon which to erect the desirable superstructure.

The following historical sketch of the British document, as given in an address by the President of the Institute, Mr. Aston Webb, on June 13th, 1903, is of much interest:

"The first time the subject was broached was on the 18th January, 1887, when Mr. T. Rickman announced at a Meeting of the Practice Committee that he had been approached by the Secretary of the Institute of Builders with a view to alterations in the existing Heads of Conditions of Contract. The Committee agreed to consider the question, and a Sub-Committee was formed. The Sub-Committee submitted their draft of General Conditions to the Practice Committee on the 18th December 1888, and for the following year the draft was considered by the Practice Committee. Negotiations were then instituted (December 1889) with the Institute of Builders. On the 29th May 1891 the Committee considered the draft as altered by the Institute of Builders, and negotiations with that Body were continued through the year. On the 9th August 1892 the first Conference between the Practice Committee and the Institute of Builders was held, the representatives of the Practice Committee being Messrs. Henry Currey, H. C. Boyes, G. E. Grayson, Edwin T. Hall, and T. M. Rickman. Negotiations then continued for two or three years, but unfortunately it was found impossible to come to an agreement. In 1895 the negotiations were dropped, and in July of that year the Institute Form of Contract, which was in use from that date to April, 1901, was issued.

"On the 5th December 1898 the Secretary of the Institute reported that he had had an interview with the Secretary of the Institute of Builders with reference to an agreement between the two Bodies on the Arbitration Clause, and a Sub-Committee of the Council, consisting of Messrs. Aston Webb, E. A. Gruning, John Slater, Wm. Emerson, Thomas Blashill and Edwin T. Hall, was appointed to confer with the Builders. Several meetings of the joint representatives were held, and on the 8th October 1900 the Institute of Builders forwarded the Report of the Joint Committee to the Council and stated that they were prepared to adopt it. On the 10th December the Council submitted the revised Paper to the General Body. The Arbitration Clause, however, as revised, met with such strong opposition that the Council referred the matter again to the Institute of Builders urging the modifications required to make the Clause acceptable to the R. I. B. A. After some further correspondence it was announced in April 1901 at a General Meeting that the negotiations between the Council and the Institute of Builders for a uniform Contract had failed, and at a Special General Meeting held on the 15th April 1901 the present Form as revised by the R. I. B. A. was sanctioned. On the 19th January 1903 the President reported to the Council that the President of the Institute of Builders had expressed the willingness of the Builders to accept the Institute Form as it now stood, with the addition of a proviso to Clause 16. The matter was referred to a Committee of the Council, who on the 18th May 1903 reported that they had conferred with Mr. King and Mr. Greenwood on behalf of the Builders, who had agreed to accept the Institute Form subject only to the modifications now proposed. The President of the Institute of Builders was then requested to send a formal communication to the Council stating that his Institute and the National Federation of Master Builders were prepared to accept the two Forms of Agreement and Conditions issued by the R. I. B. A. provided the alterations were agreed to; and that the Council would then recommend the General Body to agree to the alterations."

The present set of forms is thus adopted by both the Royal Institute of British Architects and the Institute of Builders. The debates on the various provisions of the Agreement* are well worth serious study.

Bibliographical Notes.

The following references are to a few articles or books relating to various matters affecting building agreements, and are given for the benefit of those desiring to make a more extended study of the subject:

* *Journal*, Royal Institute of British Architects, Vols. 2, 8 and 10.

"General Notes on Specifications and Contracts," by William B. Bamford, Assoc. M. Am. Soc. C. E. *The American Architect,* serial beginning Sept. 15th and ending Oct. 6th, 1909.

"The Engineer's Fault," by John C. Wait, M. Am. Soc. C. E. (A discussion of specifications, contracts, and lawsuits.) *Proceedings,* The Municipal Engineers of the City of New York, 1905. Reprint in *Engineering News,* June 9th, 1905.

"The Legal Position of Architects in Relation to Certificates and Awards," by J. A. Strahan. (A paper read before the Royal Institute of British Architects, Jan. 28th, 1895.) Abstract in *The Builder,* Vol. LXVIII, p. 82.

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"Engineers' Contracts and Specifications from a Contractor's Point of View," by James W. Rollins, Jr., M. Am. Soc. C. E. *Journal, Association of Engineering Societies,* July, 1907.

"Different Plans of Letting Contracts," by Frank B. Gilbreth. (Paper read before the American Public Works Association.) Reprint in *Engineering News,* Oct. 18th, 1906.

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"The Engineer in Construction Contracts," by John C. Wait, M. Am. Soc. C. E. *Transactions,* Association of Civil Engineers of Cornell University, 1902.

"Engineers' Specifications." (Observations on specifications in general and the legal effect of clauses of particular hardship to contractors.) *The Engineer* (London), Dec. 30th, 1904.

"The Influence of Specifications on Commercial Products," by C. B. Dudley, M. Am. Soc. C. E. (Presidential address before the American Society for Testing Materials.) *Proceedings,* American Society for Testing Materials, 1904. Reprint in *The Engineering Record,* July 9th, 1904.

"Arbitrations: A Text-Book for Arbitrators, Umpires, and All Connected with Arbitrations, more especially Architects, Engineers, and Surveyors, in Tabulated Form, with the Chief Cases Governing the Same, and an Appendix of Forms, Statutes, Rules, etc.," by Professor Bannister Fletcher, F. R. I. B. A., London (1904). Crown 8vo, 149 pages, cloth, 3d ed., revised and largely rewritten.

"Architects' Law Reports (illustrated): A Series of Revised Reports of Cases Decided in the Superior Courts, of Importance to Architects, Surveyors, and Others Interested in the Construction and Mainte-

nance of Buildings and Streets in London and the Provinces," compiled by Arthur Crow, F. R. I. B. A., Legal Editor; A. F. Jenkins, Barrister-at-law. London, Vol. I, Dec., 1904, 125 pages, 4to, paper cover; Vol. II, Dec., 1905, 275 pages, 4to, paper cover; Vol. III, Dec., 1908, 4to, paper cover.

"Building Cases: Being a Digest of Reported Decisions Affecting Architects, Surveyors, Builders, and Building Owners," by F. St. John Morrow, LL.D., Barrister-at-law. London (1906), Large 8vo.

"Building Contracts, Building Leases, and Building Statutes (English), with Precedents of Building Leases and Contracts, and Other Forms Connected with Building, and the Statute Law Relating to Building, with Notes and Cases under the Various Sections, Together with an Appendix of Unreported Building Cases," by Judge Emden. London (1907), 800 pages, 4th ed., revised, Large 8vo.

"Conditions of Contract," by Frank W. Macey. London, 1904.

"Engineering and Architectural Jurisprudence: A Presentation of the Law of Construction for Engineers, Architects, Contractors, Builders, Public Officials, and Attorneys-at-Law," by John C. Wait, M. Am. Soc. C. E. New York (1898), lxxx + 905 pages, 8vo, cloth.

"Law of Contracts," by John M. Brown (the author). Wash. (1907), 80 pages, 8vo, cloth.

"Statutory Provisions Relating to Government Contracts," by John M. Brown (the author). Wash. (1908), 49 pages, 8vo, cloth.

Divisions of Contracts.

Building contracts may be said to be divisible into four parts or divisions.

1.—Matters Preliminary to Contracts.—Being the terms under which a contract is awarded, and giving the general information necessary before the execution of the agreement. It would include the "Form of Agreement" and "Schedule of Conditions of Contract" as given hereafter.

2.—Agreement (or so-called Contract).—Being the instrument executed between the Owner and Contractor, legally and formally binding them to fulfill their obligations. It should properly include the "Schedule of Conditions of Contract."

3.—Specifications.—Being the detailed written description of the method of construction and of the quality and kind of materials to be furnished.

4.—Drawings.—Being the detailed, graphic design of the work to be done, with the quantity and sizes of materials.

This paper treats only of the Agreement (or so-called Contract).

The Agreement, preferably, should not contain any matters concerning the details of construction, these belonging properly to the specifications. There are, however, a number of legal requirements which could be included in the Agreement or in the Specification. Legally, they would be just as binding in one as in the other. These matters will be mentioned under the heading "General Conditions."

Reasons for Contracts.

A building contract is entered into by the Owner in order to obtain the services of a contractor who is skilled in that particular kind of work, and by the Contractor because he expects to make a certain profit in remuneration for his services and knowledge. Assured of his profit, it is immaterial to the Contractor what kind of work the Owner or Engineer may desire to have done, but if the Contractor's profit is not assured, he becomes vitally interested in anything that may tend to affect this prospective profit. With the Contractor, the work is purely a business proposition; he expects to make a certain profit for doing a particular amount of work. He must make a legitimate profit; his business life depends on it; and the more he feels this slipping away from him the more diligently will he find ways and means to reimburse himself. Even the best of contractors will try hard to obtain this profit in one way or another, else they must ultimately be forced from business and leave the field to those who are probably less competent contractors but shrewder business men.

A contract is intended to insure that an Owner will obtain a definite piece of work for a stated sum and within a given time, and to insure that, for doing this, a Contractor will be paid stated sums of money at definite times. The contract, therefore, should be drawn so that the Owner can demand and obtain his work and the Contractor his payments with the least delay and difficulty.

It is thought that the provisions of the Agreement and Schedule of Conditions of Contract given hereafter will protect more equitably the interests of all parties, and permit the Engineer to enforce more certainly the proper fulfillment of the contract obligations than has been possible under the majority of such documents which have come to the writer's notice.

Systems of Contracting.

At present the systems of contracting used in building work may be divided into two general classes, the lump-sum system and the various percentage systems. They are often designated by the manner in which the work is to be paid for, as:

1.—Lump sum;

2.—Cost, plus a percentage;

3.—Cost, plus a percentage—where the General Contractor takes estimates for sub-branches of work, and makes lump-sum contracts, with the Engineer's approval;

4.—Cost, plus a percentage, with guaranteed limit;

5.—Cost, plus a fixed sum;

6.—Cost, plus a fixed sum, with a bonus if cost is less than a fixed sum, and a penalty if it exceeds such sum.

Of these, the lump-sum system is the most usual, and is theoretically the best. Such a system pre-supposes that the contract defines the work to be performed, and that a contractor, therefore, can determine accurately what the execution of such work will cost. If, however, the contract requirements are indefinite, the Contractor is never certain, no matter how conscientious he may be, that he may not be required, under broad or ambiguous clauses of the contract, to furnish more than he honestly understood would be required from his reading of the contract when estimating. Thus he may be forced to do work at a net loss instead of a profit. Under the various percentage systems, on the contrary, the Contractor is assured of his profit, no matter how much work he may be required to do.

The lump-sum contract is superior to the percentage type because it more truly fosters fair and open competition, and is more economical, as has been demonstrated amply by many men of ripe experience. This presumes that the drawings and specifications describe completely and accurately everything that the Contractor must do or furnish, especially where an expense is involved. If this is not done, the Contractor's prices become mere gambles on probabilities, and not accurate estimates of actual costs plus a legitimate profit. In such a case is it to be wondered at that a Contractor will fight at every legitimate opening in order to protect his final profit? He is not always able legally to demand additional compensation for the work which

has actually cost him more than the estimated amount, therefore his only opportunity to make up his losses and come out with a fair total profit is by obtaining good prices for additional work, or, if it does not appear that there will be any such worth while, the Contractor must find flaws in the contract and endeavor to get all the extra money he can, through technicalities or otherwise.

With the irresponsible or dishonest Contractor there need be no sympathy, but by far the greater number of Contractors only desire their legitimate profit, and the prospect of making a fair profit by hard, honest work is none too bright at the present time, under the usual lump-sum contract.

Contractors have found that the easiest and most practical way to overcome the disadvantages of the lump-sum estimates is by the various percentage systems. The Contractor is thus assured of his profits, and the Owner and Engineer can change or increase the work to their hearts' content, as all additional expense involved is borne by the Owner.

Assuming that the Contractor is honest, and that the bills and statements he presents for payment are correct, and have all discounts, except a cash discount deducted, the cost of percentage work is apt to exceed the cost of work done under a lump-sum contract. The reasons for most of this increase are not hard to find. The Contractor, not being responsible for the cost, and having his profits assured, readily makes any changes and does work as directed, no matter how expensive or uneconomical such work may be, and the Owner does not realize how expensive his fads or fancies will be until the work is completed. Then again, if the percentage is based on the total cost, there is not the incentive for the Contractor to become as zealous in obtaining the closest prices for materials as he would under a lump-sum contract.

Another most serious objection to percentage work is due to the fact that mechanics are inclined to loaf, even under the most careful supervision, and it is hard to get as much work out of them as under the lump-sum system. They seem to feel that as their employer, the Contractor, will not suffer any loss, there will be no harm done, or they will not be in danger of losing their positions.

Many of the most serious objections to the percentage system have been overcome in varying degrees under Systems 3, 4, 5, and 6. These systems have much to commend them, and, with all their faults, they

will eventually supersede the lump-sum system, unless more care is taken to make contracts complete and accurate.

The English method of "Quantity Surveying" is another excellent attempt to overcome the worst objections to the lump-sum system of estimating, but, as at present in use, it would be too cumbersome for American practice. There the Owner or Engineer has the work to be done itemized in detail, and on this itemized statement, submitted to all Contractors, they fill in their prices. These lists go into matters in minute detail, and, as all Contractors estimate on the same list, the advantage is obvious. The Royal Institute of British Architects had approved forms, which are only slight modifications of the one for lump-sum contracting, in which the principal change is in making the "Quantities" a part of the contract. Under such a contract, the Contractor would be able to obtain compensation for all items not mentioned specifically in the "Schedule of Quantities."

Practical and Equitable Contracts.

Many contracts appear to be drawn by the Engineer solely on the assumption that the Owner's interests must be most carefully guarded against failure by the Contractor to fulfill his agreements properly. In the past, when profits were sufficiently liberal, Contractors could afford to take chances that "club-clauses" would not be strictly enforced, and profits lost by unfair clauses could be made up on "extras." To-day, however, competition is sharper, and profits are too often figured on too slender a margin to permit a Contractor to "take care" of losses caused by unfair clauses. No wonder, then, that the Contractor becomes more and more technical and troublesome over small omissions or errors, whereas, before, he could charge these minor matters to profit and loss, or good-will. The chances which a Contractor takes that an Owner will not be fair-minded should not be overlooked. An Owner may become very exacting, and may insist on "strict and literal" compliance with the letter of the specifications and contract, and may place an Engineer in a very embarrassing position by referring to and insisting on the literal fulfillment of certain ambiguous conditions or the "taking of every pound of flesh," against the Engineer's better judgment.

The value of a just contract to the Owner and the Engineer is as great as its value to the Contractor. A just contract, provided it is

accurate and free from ambiguity, cheapens the cost of work by eliminating the necessity for sums to cover possible contingencies; it broadens real competition, and encourages lower estimates; it ensures more rapid erection, and also acts as a powerful regulator of the mutual understanding and confidence in the fairness of all parties. Fairness, like oil for machinery, is necessary to prevent friction.

The Agreement is only one of the divisions of a contract. It is the specifications and drawings, together, which must give the Contractor a clear and complete knowledge of the work he must perform and the materials he must furnish, and should contain all the information necessary to permit him to make an accurate estimate and to carry out the work properly. The specification requirements should define clearly the result, the accomplishment of which is the object of the contract. It is a general statement of the work to be performed, a description of the materials, the quality required, and the workmanship to be performed, with definite limits as to what tests must be made in order to determine compliance with the requirements of the contract, or what defects would be sufficient cause for rejection.

Specification writing cannot be acquired successfully without practice and without broad knowledge, practical experience, and a careful study of the various materials and methods of construction and their relative costs. As ideas and methods change and improve, as a result of experience, so specifications should be changed to keep abreast of the times. An inexperienced writer is very apt to be uneven in his descriptions, writing at unnecessary length upon matters with which he is familiar, and covering his ignorance of equally important matters by the brief and ambiguous direction that "the work shall be done to the entire satisfaction of the Engineer," "whose decision is to be final and binding on all parties." These statements are too often an admission that the Engineer did not know what he desired, or that the Owner had failed to express a preference and, therefore, had been left with unlimited future choice.

As previously stated, the ultimate object of a contract is to insure to the Owner that he will obtain a definite finished product, of a certain kind, for a particular cost, and in a given time; and to insure to the Contractor that he will obtain certain sums of money at definite times for doing a definite amount of work; and the Engineer is looked upon as the arbitrator or judge, to see that both parties receive their full

justice under the contract. Contracts, therefore, should be drawn so that the Owner can demand and obtain his work and the Contractor his payments with the least delay and difficulty.

Engineers have always claimed to be arbitrators between Owner and Contractor, and not the representative solely of one party. This position is a proper one, but to-day it is more of a myth than a fact. The anomalous position is brought about, not because Engineers are desirous of being unfair, but because of the agreements (or contracts) generally used, which make it extremely hard, if not almost impossible, for Engineers to be fair if they try to conform to the agreement provisions.

It might be claimed that one of the most practical benefits to Engineers and Architects, in the more general use of an agreement similar to the one herein suggested, might be that it would not so insidiously "lead us into temptation" but would "deliver us" from many of the pitfalls lurking unsuspectingly around many present contracts.

While there is abundant room and great need for material improvement in specifications, it must be recognized that, with the limitations to the best of human endeavors, it will be extremely difficult, even for the most conscientious and painstaking, to prepare specifications for large work which may not contain some unfair requirements, when viewed from the Contractor's standpoint. We can come near to the goal, certainly much nearer than is at present the practice, but possibly our slight oversight may mean a Contractor's heavy loss.

The writer, therefore, strongly feels that one of the most practical steps toward the realization of contracts which will be fair and just to all parties will be made by the more general acceptance of the principles contained in the following forms of Agreement and Schedule of Conditions of Contract, and the adoption of similar provisions in contracts.

It is suggested that the following Agreement and Schedule of Conditions of Contract, be read through first as a whole, in order to obtain a general knowledge of their scope, omitting the reading of the notes following. They should then be read, article by article, with the comments after each.

AGREEMENT.

(For Lump-Sum Contracts.)

This Agreement, made the ..... day of ....., in the year One Thousand and Nine Hundred and ....., by and between

....., of ....., hereinafter called the Contractor, and ..... of ....., hereinafter called the Owner.

Witnesseth, that the Contractor and Owner, for the considerations hereinafter named, agree as follows:

Article I.—The Contractor agrees to provide all materials and perform all works for the.....
(here briefly describe works)
.....
.....
of a..... at .....
(here briefly describe type of building) (here briefly describe location)
shown on drawings numbered ..... and as described in the Specifications marked....., prepared by ....., hereinafter called the Engineer; and to do all things required of him by the Drawings and Specifications and subject to the Conditions set forth in the Schedule hereto annexed (hereinafter referred to as "the said Conditions").

Article II.—The Contractor agrees to complete the works by and at the time or times hereinafter stated, to wit:.....
.....
.....

Liquidated damages should be a matter for special consideration in each separate contract. Many Engineers and Owners seem to desire them, possibly for the moral effect on Contractors. The history of the legality of such provisions should cause them to be used with caution.

In many cases the following clause might be as far as it would be advisable to go toward obtaining just and reasonable damages for delay in completion:

"The Contractor agrees that the time for completion of the works shall be considered as of the essence of the contract, and he agrees that for liquidated damages he will pay the Owner for the cost of all extra inspection, and for all amounts paid for rents, or for more protracted services on the part of the Engineer, or Clerk of the Works, or both, and other expenses entailed by the Owner by delay in completing the contract, and the Owner shall be entitled to retain from the amount otherwise to become due to the Contractor an amount sufficient to reimburse properly the Owner for such expenses. Provided, however, that the sum collected as aforesaid for liquidated damages shall not exceed the sum of $..... for each and every day's delay not caused by the Owner, or otherwise as provided in the said Conditions, Section XXIV (subject to Arbitration)."

Article III.—The Owner agrees to pay the Contractor for the execution of the contract ..... dollars ($.....) in current funds subject to additions and deductions as provided in the said Conditions, and upon the presentation of the certificates of the Engineer, as follows:

Before the ..... day of each month, the Contractor may submit to the Engineer a written statement showing (*a*) the value of the work and materials actually wrought into the building to the first of the month, (*b*) the value of materials delivered at the building but not incorporated into the work, and shall deduct the aggregate of previous payments. On or before the ..... day of the same month, the Engineer shall issue a certificate for .....% of the value of work as executed in the building, until the balance retained by the Owner amounts to the sum of $....., after which time the payments shall be for the full value of the work subsequently executed. The Contractor shall be entitled, under the certificate to be issued by the Engineer, to receive payment of ....., being a part of the said sum retained in hand when the works are practically completed, and in like manner to payment of the balance within a further period of ..... months, or as soon after the expiration of such period of ..... months as the work shall have been finally completed, and all defects made good according to the true intent and meaning hereof, whichever shall last happen. The Engineer, however, shall have authority to retain out of any monies actually due the Contractor only such sums as are provided by the said Conditions, Section XXX.

The manner of payment could be varied in many ways, as the conditions of the particular case made it necessary. The principle underlying the form suggested above, however, is important.

As before stated, it is as essential that the Contractor shall be able to demand and obtain his payments as that the Owner shall be able to obtain the work for which he pays.

In Article III the Contractor's right to definite payments is expressed clearly. The Engineer must issue a certificate for money due, and the Owner must pay the amount due. It should be noted that the Engineer is allowed ample discretionary power to withhold any money not justly due, as provided by Section XXX of the Conditions. Thus the obligations become fairly clean cut. If the Contractor's work is not done properly, there is ample power to withhold money to cover definite defects. On the other hand, if the Engineer or Owner withhold any payments without just cause, the Contractor has recourse to

Arbitration, under Section XXXIV of the Conditions, or to terminate the contract, under certain contingencies, as provided in Section XXXIII.

The idea of retaining percentages until the completion of the work is intended to insure its proper completion. The usual American practice of retaining an unchanging percentage from start to finish is unnecessarily extravagant, and in contracts involving large amounts is more harmful than beneficial. After the work has been approximately half completed, the remainder should be paid for in full without retaining a percentage. Of course, additional sums can be retained to cover known defects. The sum retained when the work is half completed is usually sufficiently liberal to cover possible contingencies, and is just as effectual as to continue to increase it until completion.

Article IV.—It is mutually agreed that the term "the Engineer" in the said Conditions shall mean the said .....or in the event of his death or ceasing to be the Engineer, for the purpose of the contract, such other person as shall be nominated for that purpose by the Owner, not being a person to whom the Contractor shall object for reasons considered to be sufficient by the Arbitrators or Arbitrator mentioned in the said Conditions. Provided, always, that no person subsequently appointed to be Engineer under this contract shall be entitled to disregard or overrule any decision, or approval, or direction, given or expressed by the Engineer for the time being.

The purpose of Article IV is to cover cases which may be unusual but are generally very troublesome when they occur. It would prevent a subsequent Engineer from altering at will matters of personal opinion or choice of his predecessor. No two minds are exactly alike, and few Engineers or Architects have exactly similar tastes. Should an Engineer desire to change work which may be purely matters of opinion or judgment of both himself and his predecessor, such changes should not be paid for by the Contractor, but by the Owner, if he decides to follow the suggestions of his adviser. Of course, defective work should be condemned at any time, when discovered. Should an Engineer desire to change some orders of his predecessor, and take the ground that certain work should be done as a part of the contract, he should not have arbitrary power to enforce his will, but could order the work done as provided by Section XVII of the Conditions, leaving to the

Contractor the right of appeal to arbitration, under Section XXXIV, if he desires.

Article V.—It is mutually agreed that the said Conditions shall be read and construed as forming a part of this Agreement, and the parties hereto will respectively abide by and submit themselves to the conditions and stipulations, and perform the agreements on their parts, respectively, in such Conditions contained.

Article VI.—The Contractor and Owner, for themselves, their successors, executors, administrators, and assigns, hereby agree to the full performance of the covenants and agreements herein contained.

In Witness Whereof, the Contractor and Owner have hereunto set their hands and seals, the day and year first above written.

In presence of

.... [SEAL]
.... [SEAL]

SCHEDULE OF CONDITIONS OF CONTRACT.

I.—Drawings and Specifications.—The works shall be erected in accordance with the directions and to the reasonable satisfaction of the Engineer, in accordance with the Contract Drawings and Specifications, and in accordance with such further drawings, details, instructions, directions, and explanations as may from time to time be given by the Engineer. If the work shown on any such further drawings or details, or explanations be, in the opinion of the Contractor, extra to that comprised in the Contract, he shall, before proceeding with such work, give notice in writing to this effect to the Engineer. In the event of the Engineer and Contractor failing to agree as to whether or not there is any extra, and of the Engineer deciding that the Contractor is to carry out the said work, the Contractor shall accordingly do so, and the question whether or not there is any extra, and if so the amount thereof, shall, failing agreement, be settled by Arbitration, as provided in Section XXXIV, and the Contractor will be paid accordingly.

This section covers the troublesome question of work shown by details, etc., being in excess of the contract requirements. It might be reasonable to assume that the Engineer would know better than anyone else whether his details require extra work, and the Contractor should be permitted at any time to recover the value of such work upon proving that it was extra to the contract. There are many sound objections to such a course, and the more satisfactory of the two evils would seem to be to require the Contractor to notify the Engineer

of any extra work, before proceeding with it, as a precedent to obtaining extra compensation.

II.—Contractor to Provide Everything Necessary.—The Contractor shall provide everything necessary for the proper execution of the works, within the limits established by the Drawings and Specifications, according to the true intent and meaning of the Drawings and Specifications taken together, whether the same may or may not be particularly shown on the Drawings or described in the Specifications, provided that the same is reasonably to be inferred therefrom; and if the Contractor finds any discrepancy in the Drawings, or between the Drawings and Specifications, he shall immediately refer the same to the Engineer, who shall decide which shall be followed. Figured dimensions shall be followed in preference to the scale, and figures on large-scale drawings shall take precedence over those at smaller scale.

III.—Copies of Drawings and Specifications.—The Contract Drawings and Specifications shall remain in the custody of the Engineer, and shall be produced by him at his office as and when required by the Contractor or the Owner. Three complete copies of all Drawings and of the Specifications shall be furnished by the Engineer, free of cost, to the Contractor, for his own use. The Engineer shall furnish to the Contractor within ..... days after the receipt by him of a request for the same, any details which, in the opinion of the Engineer, are necessary for the execution of any part of the work, such request to be made only within a reasonable time before it is necessary to execute such work in order to fulfill the contract. A complete set of such copies and details shall be kept on the works until the completion thereof, and the Engineer or his representative shall at all reasonable times have access to the same, and, as instruments of service, all drawings are the property of the Engineer and shall be returned to the Engineer by the Contractor at the time of making the final payment under the contract.

IV.—Conform to Laws, Regulations, Etc.—The Contractor shall conform to the provisions of all the State building and sanitary laws, and with all municipal or local ordinances, rules, and regulations relating to building and to the preservation of the public health and safety, also with all regulations of the Board of Fire Underwriters and Fire Insurance Authorities having jurisdiction, and of any Water and Lighting Companies with whose systems the structure is proposed to be connected, and, before making any variations from the Drawings and Specifications that may be necessary by so conforming, shall give to the Engineer written notice, specifying the variation proposed to be made, and the reason for making it, and apply for instructions thereon. In case the Contractor shall not in due course receive such instructions,

he shall proceed with the work, conforming to the provisions, ordinances, rules, and regulations in question, and the variation thus necessitated shall be dealt with under Section XXXIV. All contract requirements over, above, and beyond the said provisions, ordinances, rules, and regulations, shall be fully complied with. The Contractor, at his own expense, unless the law specifically requires the Owner to do so, shall obtain all necessary permits and licenses, give all necessary notices, and pay all legal fees.

The laws, ordinances, etc., of States and municipalities are often very divergent in their requirements, and in places positively contradictory. The meaning of many of their requirements, together with a great many of those of various Fire Insurance Authorities, is not easy to determine by the most careful reading. Then again, the local authorities may have discretionary powers, and their "personal preferences" may be difficult to obtain, even by local men. Thus it would appear that, if an Engineer cannot interpret correctly the local requirements in his drawings and specifications, it is unfair to place the responsibility, by a blanket clause, on the Contractor. If, therefore, the authorities require work to be done in a different manner than that called for by the contract documents, and such work involves additional expense, the Owner, and not the Contractor, should pay for the same.

V.—Setting Out Work.—The Contractor shall set out the work in accordance with the Owner's survey and the Engineer's drawings, and, during the progress of the building, shall correct at his own expense any errors arising from inaccurate setting out, unless the Engineer shall decide to the contrary.

It is advisable that the Owner have a survey made, and guaranteed, if desired, by some recognized Title Guaranty Company. With this survey and the drawings, the Contractor should be held to lay out the work correctly; but, to renounce all responsibility for the survey or drawings, and endeavor to place the responsibility for all conditions, both known and unknown, on a Contractor, is not good business policy, nor keen professional foresight, though there may be specific cases apparently to justify such a course.

VI.—Foreman.—The Contractor shall keep constantly on the works a competent general foreman, and any directions or explanations given by the Engineer to such foreman shall be held to have been given to the Contractor.

VII.—Assignment of Contract.—The Contractor shall not, without the written consent of the Engineer, assign this agreement, and, before subletting any portion of the works, shall notify the Engineer of such fact, in writing. No sub-contractors, to whom the Engineer has made written objections, shall be employed, and the Contractor shall assume full responsibility for all his sub-contractors.

Sub-contractors are not legally recognized as parties to a contract, but, in many cases the power is assumed of dictating to a Contractor what persons he must employ as sub-contractors. Such a course is not only unfair, but might cause legal difficulties later, should the Contractor take the ground that he is not responsible for the acts of a sub-contractor, not of his own selection, but who was forced upon him by the Engineer or Owner. In the foregoing section the Engineer may disapprove of sub-contractors who are not satisfactory, but he is not permitted to dictate who must be employed as sub-contractors. Where the Engineer approves a sub-contractor, it can be implied that he considers that the person whom he may approve will be satisfactory, while, by confining himself to the disapproval of unsatisfactory sub-contractors, no such satisfaction can be implied, as the persons finally employed may be entirely unknown to the Engineer.

VIII.—Work by Other Contractors.—The Contractor shall permit the execution of work by any other mechanics or trades employed by the Owner, and shall afford them every reasonable facility for the proper erection of their work. The Owner agrees not to employ any other mechanics or trades on the work against whom the Contractor shall make what the Engineer considers a reasonable objection, or who will not enter into a contract with the Contractor guaranteeing the due performance of his work and indemnifying the Contractor against valid claims which may arise during the performance of the work.

The object of this section is simply to minimize the chances for disagreements between independent contractors and their making claims to the Owner for damages done to them by other contractors. By this section, if a General Contractor is responsible for the completion of certain work, he is permitted to make reasonable objection to other independent Contractors before they are employed, and to exact a warranty for valid claims arising out of the work. If they agree to indemnify each other, the Owner has then fulfilled his obligations; then any disagreements must be settled between the individual Con-

tractors, and the Owner can demand with a clear conscience that they shall perform their contract obligations, and settle their own differences.

IX.—Materials, Etc., to Conform to Specifications.—All materials and workmanship shall be of the respective kinds described in the Specifications, and the Contractor, upon the request of the Engineer, shall furnish him with vouchers to prove that the materials are such as are specified. The Contractor shall also furnish duplicate labelled samples of materials and workmanship, for the Engineer's approval, and the materials and workmanship furnished shall be equal to the approved samples.

X.—Dismissal of Workmen.—The Contractor, on the written request of the Engineer, shall immediately dismiss from the works any person employed thereon by him, who, in the opinion of the Engineer, may be incompetent, or who misconducts himself, and such person shall not be again employed on the works without the permission of the Engineer.

XI.—Access to Work.—The Engineer and any person authorized by him shall at all reasonable times have access to the works, and the Engineer and his representatives shall at like times have access to the workshops of the Contractor or other places where work is being prepared for the building.

XII.—Clerk of Works.—The Clerk of the Works or other representative shall be considered to act solely as inspector, and under the Engineer, and the Contractor shall afford him every facility for examining the works and materials. The Clerk of the Works or other representative shall have no power to revoke, alter, enlarge, or relax any requirements of the contract, except in so far as such authority may be specifically conferred by a written order of the Engineer.

XIII.—Work to Be Opened Up.—The Contractor, at the request of the Engineer, and within such times as the Engineer shall name, shall open for inspection any work covered up; and should the Contractor refuse or neglect to comply with such request, the Engineer may employ other workmen to open up the same. If the said work has been covered up in contravention of the Engineer's instructions, or if, on being opened up, it is found to be not in accordance with the Drawings and Specifications or the instructions of the Engineer, the expense of opening and covering it up again, whether done by the Contractor or such other workmen, shall be borne by, and shall be recoverable from, the Contractor, or may be deducted, as hereafter provided. If the work has not been covered up in contravention of such instructions, and be found in accordance with the said Drawings and Specifications or instructions, then the expense aforesaid shall be borne by the Owner and be added to the contract sum: provided always that in the case of foundations, or other urgent work thus opened up and requiring immediate attention,

the Engineer, within a reasonable time after the receipt of notice from the Contractor that the work has been so opened, shall make or cause the inspection thereof to be made, and at the expiration of such time, if such inspection shall not have been made, the Contractor may cover up the same, and shall not be required to open it up again for inspection except at the expense of the Owner.

This section seems to indicate the fairest practical way of solving a perplexing problem.

XIV.—Removal of Work.—During the progress of the works, the Engineer shall have power to order in writing from time to time the removal from the works, within such reasonable time or times as may be specified in the order, of any materials which in the opinion of the Engineer are not in accordance with the Specifications or the instructions of the Engineer, the substitution of proper materials, and the removal and proper re-execution of any work executed with materials or workmanship not in accordance with the Drawings and Specifications or instructions; and the Contractor shall forthwith carry out such order at his own cost. In case of default of the Contractor to carry out such order, the Owner shall have power to employ and pay other persons to carry out the same; and all expenses consequent thereon or incidental thereto shall be borne by the Contractor, and shall be recoverable from him by the Owner, or may be deducted by the Owner from any monies due or that may become due to the Contractor. If, in the opinion of the Engineer, it is not expedient to remove materials delivered or alter work done which is not in accordance with the contract, the Owner may deduct the difference in value of the materials as delivered and the work as done and that required by the Drawings and Specifications, together with a fair amount for damages, which sum shall be determined by the Engineer (subject to Arbitration).

XV.—Defective Work.—Any defects, shrinkage, or other faults which may appear within.....months from the completion of the works, and, in the opinion of the Engineer, arising from materials or workmanship not in accordance with the Drawings and Specifications or the instructions of the Engineer, or any damage to pointing by frost appearing within the like period, shall, upon the direction in writing of the Engineer, and within such reasonable time as shall be specified therein, be amended and made good by the Contractor at his own expense, unless the Engineer shall decide that he ought to be paid for the same; and, in case of default, the Owner may employ and pay other persons to amend and make good such defects, shrinkage, or other faults or damage, and all expenses consequent thereon or incidental thereto shall be borne by the Contractor and shall be recoverable from him by the Owner from any monies due or that may become due to the Contractor. Should any defective work have been done

or should any defective materials be supplied by any sub-contractor employed on the work who has been nominated or approved by the Engineer, as provided in Sections VII and XXIX, the Contractor shall be liable to make good in the same manner as if such work or materials had been done or supplied by the Contractor and had been subject to the provisions of this and the preceding Section.

XVI.—Copy of Quantities, Etc.—The Contractor, if required before executing this agreement or before making application for any payment, shall furnish to the Engineer, upon a blank form prepared by him, a correct, verified, itemized statement, showing the quantities of labor and materials that will be required to perform the principal items of the contract, together with their prices; and the amount included in his estimate for each general item of work to be furnished complete and in position, including in each item its due proportion of expense and profit, the sum of the different items equalling the total amount of the proposal. The above schedule shall be for the use of the Engineer, at his discretion, only in preparing estimates for payments on account to the Contractor.

If this section is read in connection with Sections XVII and XVIII, it will be seen that possibly many apparently desirable modifications are covered by the provisions of those two sections.

XVII.—Variations and Extras.—The Contractor, when authorized by the Engineer, or as provided by Section IV, shall vary by altering, adding to, or deducting from the contract requirements; such authorization is to be sufficiently proven by any writing or drawing signed by the Engineer, or by any subsequent written approval by him, but the Contractor shall make no variation without such authorization. No claim for extra compensation will be allowed unless it shall have been executed under the provision of Section IV, or by the authority of the Engineer, as herein mentioned. Such an extra is hereinafter referred to as an authorized extra.

Variations in the contract (or extras) is one of the most troublesome phases of lump-sum contract work. By the provisions of this section and Section XVIII, with the final redress to arbitration under Section XXXIV, it is hoped, that the greater part of such controversies can be adjusted satisfactorily.

XVIII.—Price for Extras.—No variation shall vitiate the contract; but all alterations, additions to, or deductions from the contract requirements, which may have been made with the authority of the Engineer, or without his authority, provided he gives subsequently a written sanction to such omission, shall be valued in one of the four following ways, as may be selected by the Owner:

A.—By Estimate and Acceptance of a Lump Sum. The Contractor shall submit an estimate for making the variation contemplated, and in case an agreement as to price cannot be reached, the Engineer may order the work to proceed and the Contractor shall forthwith proceed with such variation and leave the price to be settled by Arbitration. A tentative amount, however, will be allowed by the Owner, equal to the proportion of the amount of the estimate conceded by both the Owner and Contractor, and payments will be made on the basis of such amount "without prejudice" until the correct amount shall be determined by Arbitration.

B.—By Unit Prices. Where the Contractor shall have given, in his estimate for the contract, or in subsequent accepted estimates, unit prices for specified parts of the work, the Engineer may make an award of a sum based on these unit prices, subject to Arbitration.

C.—By Net Cost. In this case the Contractor will be paid the actual net amount paid by him for materials; after all trade discounts, except a *bona fide* cash discount, have been deducted; plus the cost of labor or material required to erect and finish the work properly; plus ....% for profit, general office superintendence, and other general office expenses not peculiar to this contract; plus ....% per month on the actual cost for reproducing the plant as used for such extra work, for plant rental. The total net cost shall not exceed the usual reasonable cost for similar work, unless the Contractor can prove conclusively that the excess was due to exceptional and unavoidable difficulties or conditions. At required intervals, of not less than seven days, the Contractor shall render detailed statements and vouchers, and the Engineer will award an amount as the net cost, subject to Arbitration. In variations involving the omission of work, the amount to be deducted for such omission shall be based on the net cost as given above, plus ....% for prospective profit.

D.—By Cost Plus a Lump Sum. In such cases the Contractor shall submit an estimate which he agrees the variation shall not exceed, plus a sum for profit. The Contractor, at required intervals, of not less than seven days, shall render detailed statements and vouchers of the net cost, on the same basis as given under *"C"* above, except that no profit shall be included. If the actual cost is less than that given in his estimate, the Contractor shall receive, in addition to the lump sum profit, ...% of the difference between the actual cost and the amount of his estimate. If the actual cost is greater than that given in his estimate, the Contractor shall receive, in addition to the amount of his estimate, only ....% of the difference between the actual cost and the amount of his estimate.

The foregoing methods of determining the price to be paid for variations to the contract (or extras) with the redress to Arbitration

(Section XXXIV), is in principle the most equitable method that has come to the writer's notice, for solving one of the most vital parts of a contract—the proper compensation for legitimate extra work. This section, in detail, is entirely different from anything which the writer has seen in use, and he suggests for it special consideration. In principle, it is similar to the requirements of the Royal Institute of British Architects.

XIX.—Damage to Persons and Property.—The Contractor shall indemnify and save harmless the Owner from any and all claims and demands which may be made by reason of (1) any injury or damage suffered or sustained by any person or corporation, caused by, or alleged to have been caused by, any act or omission of the Contractor or his agents, servants, workmen, employees, or sub-contractors, in the course of the performance of the work; and (2) any injury or damage suffered or sustained by any such agents, servants, workmen, employees, or sub-contractors, however caused, in or about said building; and the Contractor, at his own cost, expense, and risk, shall defend any and all actions, suits, or other legal proceeding that may be brought or instituted against the Owner on any such claim or demand, and pay and satisfy any judgment that may be rendered against the Owner in any such action, suit, or other legal proceeding, or result thereof; provided, however, that this section shall not be construed as an assumption by the Contractor of any liability to the Owner for any injury or damage caused by an employee of the Owner who is not a Contractor or sub-contractor, or an agent, servant, workman, or employee of a Contractor or sub-contractor engaged in the performance of work in or about said building.

XX.—Damage to Work.—The Contractor shall assume the responsibility for, and repair at his own expense, all damage to his work, or the work of other Contractors, or to the building and contents, caused by the installation of work or by the carelessness or negligence of himself, his sub-contractors, or employees. He shall also be responsible for, and make good, any damage to adjoining property, caused by any movement or settlement, or by material or implements falling upon the same during construction, or for any other cause for which the Owner may be held liable, and shall thoroughly complete the whole of the works.

XXI.—Patent Infringement, Etc.—The Contractor shall indemnify and save harmless the Owner from any and all claims and demands which may be made by reason of any infringement, or alleged infringement, of any patent, patent rights, or claim, caused, or alleged to have been caused, by the use of any apparatus or appliances, or portions thereof, furnished or installed by the Contractor; and the Contractor,

at his own cost, expense, and risk, shall defend any and all actions, suits, or other legal proceedings that may be brought or instituted against the owner on any such claim or demand, and pay and satisfy any judgment or decree that may be rendered or adjudged against the Owner in any action, suit, or other legal proceeding or result thereof.

XXII.—Insurance.—During the progress of the works, the Owner will maintain insurance on said works, in his own name and in the name of the Contractor, against loss or damage by fire or lightning, and the policies will cover all work incorporated in the building, except that of excavation and foundations below the level of the ground, and all materials for same in or about the premises, and be made payable to the Owner, for adjustment between the parties hereto, as their interests may appear. At the end of each month the Contractor shall deliver to the Owner a statement showing the total value of work done and materials delivered, as herein described, to that date, and the owner will maintain said insurance in accordance with the latest statement thus delivered.

The objection to most insurance clauses is the difficulty of adjusting properly the interests as they may appear. Fire Insurance Companies usually hold such payments until both parties sign an agreement stating the proportionate amounts each party is to receive. This adjusting, in case of a disagreement, may consume considerable time, and delay the continuance of the work. Under Section XXII the amount of the insurance is payable directly to the Owner, and as he receives monthly a statement from the Contractor and insures the Contractor in accordance with such statement, it is fairly simple to estimate the Contractor's loss at any time. If the Owner neglects to insure himself properly, he then has no one but himself to blame.

XXIII.—Damages for Delay.—Should the Owner claim damages for delay in the completion of the works, the Engineer, if, in his opinion, any damages be payable, will make an award on the basis stated in the Agreement, and will write the amount across the face of the final certificate as "the amount to be deducted from the face hereof for delay in completion of the work" (subject to Arbitration). Thereupon the Engineer's services in this matter shall terminate, except as provided under Section XXXIV.

Should the Contractor, by his own fault or negligence, unnecessarily delay the completion of the work, thereby necessitating unusual or more protracted services on the part of the Engineer or the Clerk of the Works, or both, the Owner shall be entitled to retain from the amount otherwise to become due the Contractor an amount sufficient

to reimburse properly either the Engineer or the Clerk of the Works, or both, for such protracted or unusual services (subject to Arbitration).

This section should be read in connection with any damages to be allowed under Article II of the Agreement. It should be distinctly stated in the Agreement whether or not the specified sum for damages includes the protracted services of the Engineer, etc., herein mentioned. In many cases compensation for such protracted services might be all the damage that should justly be demanded.

XXIV.—Extension of Time.—If, in the opinion of the Engineer, the work be delayed by *force majeure* or by reason of any exceptionally inclement weather, or by reason of instructions from the Engineer in consequence of proceedings taken or threatened by, or disputes with, adjoining or neighboring owners, or by the work or delay of other Contractors or mechanics engaged or nominated by the Owner or the Engineer, and not referred to in the Specifications, or by reason of authorized extras or additions, or in consequence of any notice reasonably given by the Contractor in pursuance of Section IV, or by the combined action of workmen, in no wise caused by or resulting from default or collusion on the part of the Contractor, or in consequence of the Contractor not having received in due time necessary instructions from the Engineer for which he shall have specifically applied in writing, the Engineer will make a fair and reasonable extension of time for completion in respect thereof. In case of such combined action of workmen, the Contractor shall, as soon as may be, give to the Engineer written notice thereof; but the Contractor shall, nevertheless, use his best endeavors to prevent delay, and shall do all that may reasonably be required, to the satisfaction of the Engineer, to proceed with the works.

The usual practice is to grant an extension of time to a Contractor equal to the time which he may be delayed, provided he gives notice of such delay within 48 hours. The reasons for which extensions may be granted are usually very few, and not always of the most importance. The section suggested above will probably be objected to as not definitely specifying the length of the extension of time that may be allowed. The Engineer is empowered to make a fair and reasonable extension of time for completion, and he can thus take into account matters which it would be hard to cover by an arbitrary clause. Should either party feel aggrieved by the length of time of the extension, it has the right of appeal to Arbitration (Section XXXIV).

XXV.—Suspension of Work by Contractor.—If the Contractor, except on account of any legal restraint upon the Owner preventing the continuance of the works, or on account of any of the causes mentioned in Section XXIV, or in case of any certificate being withheld or not paid when due, shall suspend the works, or, in the opinion of the Engineer, shall neglect or fail to proceed with due diligence in the performance of his part of the contract, or if he shall more than once make default in the respects mentioned in Section XIV, the Owner, by the Engineer, shall have power to give notice in writing to the Contractor requiring that the works be proceeded with in a reasonable manner and with reasonable despatch. Such notice shall not be unreasonably or vexatiously given, and must signify that it purports to be a notice under the provisions of this section, and must specify the act or default on the part of the Contractor upon which it is based. After such notice shall have been given, the Contractor shall not be at liberty to remove from the site or works, or from any ground contiguous thereto, any plant or materials belonging to him which shall have been placed thereon for the purpose of the works; and the Owner shall have a lien upon all such plant and materials, to subsist from the date of such notice being given until the notice shall have been complied with. Provided always that such lien shall not, under any circumstances, subsist after the expiration of thirty-one days from the date of such notice being given, unless the Owner shall have entered upon and taken possession of the works and site, as hereinafter provided. If the Contractor shall fail for .... days after the time given in such notice to proceed with the works as therein prescribed, the Owner shall be at liberty to provide any such labor or materials, and to deduct the cost thereof from any money then due or thereafter to become due to the Contractor under the contract.

The general purport of this section is similar to the majority of similar American clauses. Its wording, however, is freer from uncertainties, and deserves careful attention.

XXVI.—Termination of Contract by Owner.—If the Contractor shall fail for .... days after such notice has been given as provided in Section XXV, to proceed with the work as therein prescribed, and if the Engineer shall then certify that such refusal, neglect, or failure is sufficient ground for such action, the Owner may enter upon and take possession of the works and site, and of all plant and materials thereon (or on any ground contiguous thereto), intended to be used for the works, and all such materials as above mentioned shall thereupon become the property of the Owner absolutely, and the Owner shall retain and hold a lien upon all such plant until the works shall have been completed under the powers hereinafter conferred on him. If the

Owner shall exercise the above power he may engage any other person to complete the works, and exclude the Contractor, his agents, and servants, from entry upon or access to the same, except that the Contractor or any one person nominated by him may have access at all reasonable times to inspect, survey, and measure the works. And the Owner shall take such steps as, in the opinion of the Engineer, may be reasonably necessary for completing the works without undue delay or expense, using for that purpose the plant and materials above mentioned, in so far as they are suitable and adapted to such use. Upon the completion of the works, the Engineer shall certify the amount of the expense properly incurred consequent on and incidental to the default of the Contractor as aforesaid, and in completing the works by other persons. Should the amount thus certified as the expenses properly incurred be less than the amount which would have been due to the Contractor upon the completion of the work by him, the difference will be paid to the Contractor by the Owner; should the amount of the former exceed the latter, the difference shall be paid by the Contractor to the Owner. The Owner will not be liable to make any further payment or compensation to the Contractor for or on account of the proper use of the plant for the completion of the works under the provisions hereinbefore contained other than such payments as are included in the contract price. After the works shall have been thus completed by persons other than the Contractor, under the provisions hereinbefore contained, the Owner shall give notice to the Contractor of such completion, and may require him from time to time, before and after completion, to remove his plant and all such materials as aforesaid as may not have been used in the completion of the works, from the site. If such plant and materials are not removed within a reasonable time after notice shall have been given, the Owner may remove and sell the same, holding the proceeds, less the cost of their removal and sale, to the credit of the Contractor. Any notice to be given to the Contractor under this clause shall be given by leaving the same at the place of business of the Contractor, or by a registered letter sent to him at that address.

This section conforms in substance to the generally accepted American practice, but its wording is more definite, and it leaves fewer loopholes for argument or controversy.

XXVII.—Net Cost.—The words, "Net Cost," or the initials, "N. C.," used in the Specifications, for materials to be obtained and installed by the Contractor, shall mean, unless otherwise stated in the Specifications, the sum paid to merchants, after deducting all trade discounts, for such materials, in the ordinary course of delivery, but not deducting discount for cash, and such sum shall be exclusive of special cartage, the cost of installation, and Contractor's profit.

XXVIII.—Provisional Sums.—The provisional sums mentioned in the Specifications for materials to be supplied or for work to be performed by special contractors or mechanics, or for other works or fittings to the building, shall be paid and expended at such times, and in such amounts, and to and in favor of such persons, as the Engineer shall direct, and sums thus expended shall be payable by the Contractor without deduction or discount, or (without prejudice to any rights of the Contractor existing under the contract referred to in Section XXIX) by the Owner to the said Contractors or mechanics. The value of works which are executed by the Contractor in respect of provisional sums, or in additional works, shall be ascertained as provided by Section XVIII. At the settlement of accounts the amount paid by the Contractor to the said contractors and mechanics, and the said value of such works executed by the Contractor, shall be set against all provisional sums or any sum provided for additional works, and the balance, after allowing *pro rata* for the Contractor's profits at the rate of ....%, unless different rates are contained in the Contractor's original estimate, shall be added to or deducted from the contract sum, provided that in estimating the amounts paid as last herein provided no deductions shall be made by or on behalf of the Owner in respect of any damages paid by the sub-contractor to the Contractor, the intention being that the Contractor and not the Owner shall have the benefit of any such damages.

There are many cases where important matters may not be definitely determined before the execution of a contract. The practice in many cases is to insert a blanket clause giving almost unlimited selection to either Owner or Engineer. In such case a Contractor must make a shrewd guess at the amount to include in his estimate. Or something as nearly as possible to what may be required may be specified, and afterward the problem is to reconcile the cost of the work as specified with the work as required. The idea of having Provisional Sums inserted in a contract to cover matters not distinctly determined, is not new, but the above section may suggest a wording which will overcome many of the previous objections to such Provisional Sum clauses.

XXIX.—Sub-Contractors.—All contractors, merchants, mechanics, or others executing any work, or supplying any materials for which net cost prices or provisional sums are included in the Specifications, who may at any time be nominated, selected, or approved by the Engineer are hereby declared to be sub-contractors employed by the Contractor; but no sub-contractor shall be employed upon the works against whom the Contractor shall make what the Engineer considers reasonable objections, or who shall not enter into a contract with the

Contractor upon terms and conditions consistent with those in this Contract, and securing the due performance and maintenance of the works supplied or executed by such sub-contractors, and indemnifying the Contractor against any claims arising out of the misuse by the sub-contractor or his workmen of any scaffold erected or plant used by the Contractor, or that may be made against the Contractor in consequence of any act, omission, or default of the sub-contractor, his servants or agents, and against any liability, as given under Sections XIX, XXI, or XXII.

This section is intended to cover cases where it becomes desirable to have special Contractors execute work covered by Provisional Sums. In many cases particular types or makes of articles cannot be obtained excepting from special parties. To require a Contractor to employ such parties and not be relieved from responsibility for their actions, is a difficult problem. This section will prevent a Contractor, who may be under heavy penalty to complete his work, from being compelled to employ a particular Contractor for a sub-branch, who may absolutely refuse to indemnify the Contractor if he fails to complete his work on contract time, or to give a warrant for the faithful performance of the work.

XXX.—Payment and Certificate.—The Engineer shall issue his certificates for payments due the Contractor in accordance with the terms of the Agreement, provided, however, that the Engineer may withhold only such proportion of any sum due as in his judgment will cover the expense of the liens as given under Section XXXI, or of correcting such violation as given in Section XIV, and such payments shall not become due until violations are properly corrected (subject to Arbitration) or the liens are removed as hereinafter provided. No certificate given or payment made under the contract, nor partial or entire occupancy of the building by the Owner, shall be construed as an acceptance of defective work, or of improper materials, or as condoning any omission, provided, however, that in the case of a partial or entire occupancy of the building by the Owner, the Contractor shall have been given notice of all defects or omissions reasonably noticeable before such occupation. No certificate, final or otherwise, shall be construed as relieving the Contractor from his obligations to make good any defects discovered in his work after the completion and acceptance of the same, nor as a waiver of any specific obligation the Contractor may have assumed as to the durability of his works. The acceptance by the Contractor of the payment of the final certificate shall constitute a waiver of all claims against the Owner under or arising

out of the contract. The Contractor, before each payment falls due, shall submit to the Engineer a requisition therefor and shall, if required, submit therewith an itemized statement of the quantities and value of works in general items performed to date, and such statements shall be made in form provided or approved by the Engineer.

The provisions of this section are of great importance in any contract. Its importance is in its wording, and not because it contains any new principle.

XXXI.—Liens.—If at any time there shall be evidence of any lien or claim, for which, if established, the Owner or the premises might be made liable, and which would be chargeable to the Contractor, the Owner shall have the right to retain out of any payment then due, or thereafter to become due, an amount sufficient to indemnify himself against such lien or claim until the same shall be effectually discharged, or bonded; and should there prove to be any such claim after all payments are made, the Contractor shall refund to the Owner all moneys that the latter may be compelled to pay in discharging any lien upon said premises made obligatory in consequence of the Contractor's default. Before the final payment shall become due, the Contractor shall deliver to the Owner a satisfactory release of all liens against the premises on the part of all persons who have delivered materials for use in, or performed work on, said premises, including a release from the Contractor himself.

XXXII.—Unfixed Materials.—When the Contractor shall have received payment of any certificate in which the Engineer shall have stated that he has taken into account the value of any unfixed materials intended for the works, and placed by the Contractor thereon, or upon ground adjacent thereto, all such materials shall become the property of the Owner, and shall not be taken away, except for the purpose of being used on the building, without the written authority of the Engineer; and the Contractor shall be liable for any loss of or damage to such materials.

XXXIII.—Termination of Contract by Contractor.—Should the Owner not pay the Contractor any sum certified by the Engineer within the times respectively named in the Agreement, the Contractor shall give to the Owner written notice of the non-payment, and should the Owner not pay any such sum within the period of .... days from the date of delivery of such notice at the Owner's address, or sent to him there in the ordinary course of mail by registered letter, or if the Owner shall become bankrupt or file any petition for liquidation of his affairs, and if his Receiver in Bankruptcy shall repudiate this contract, or if the Receiver shall be unable to show, within .... days, to the reasonable satisfaction of the Contractor, his ability to carry out

the contract, and to make all payments due or to become due thereunder, or if the works be stopped for .... months, under an order of the Engineer or any Court of Law, the Contractor shall be at liberty to terminate the contract by notice in writing to the Engineer, and to recover from the Owner payment for all work executed and for any loss he may sustain upon any plant or material supplied or purchased for the purpose of the contract. In ascertaining the amount of such payment, the rates contained in the Contractor's original estimate shall be followed, or, where the same may not apply, rates proportionate to the prices therein contained, as far as the same are applicable.

Few American contracts contain any provision permitting a Contractor to terminate a contract on any condition whatsoever. In fact, most contracts are filled with clauses designed to bind a Contractor hand and foot so that, no matter what may happen, he is certain to remain a party to the contract. The absolute unfairness of many such requirements is hard to account for, except that they have been inherited from times when work and conditions were totally different from those existing to-day. The fairness of Engineers including within contracts a section similar to the above should be apparent from a careful reading of the section.

XXXIV.—Arbitration.—Provided always that in case any dispute or difference shall arise between the Owner, or the Engineer on his behalf, and the Contractor, either during the progress of the works or after the termination, abandonment, or breach of the contract, as to the construction of the contract or as to any matter or thing arising thereunder (except as to matters left to the sole discretion of the Engineer under Sections II, VII, X, XIV, and the exercise by him, under Section XIII, of the right to have any work opened up), or as to the withholding by the Engineer of any certificate to which the Contractor may claim to be entitled, then either party shall forthwith give to the other notice of such dispute or difference, and such dispute or difference shall be and is hereby referred to the arbitration and final decision of three disinterested arbitrators, one chosen by the Owner, one by the Contractor, and the third chosen by these two arbitrators. The decision of any two of these arbitrators shall be final and binding on both parties hereto. If the parties hereto mutually agree, one arbitrator may be appointed in place of the three named as above, and his decision shall be final and binding on both parties.

When the time for submitting matters to arbitration, as herein provided, has arrived, either the Owner or the Contractor may appoint an arbitrator and demand the appointment of an arbitrator by the other party. Should the party filing notice of appeal to arbitration as

herein provided fail to choose an arbitrator or to agree on a single arbitrator within seven days after receiving a written request from the other party, his right to arbitration in such matters shall lapse; should the other party fail to choose an arbitrator or to agree on a single arbitrator within seven days after receiving a written request so to do, then such dispute or difference shall be and is hereby referred to the arbitration and final decision of ....., or in the event of his death, or unwillingness or inability to act, of ....., or in the event of his death, or unwillingness or inability to act, of a person to be appointed on the request of either party after giving notice to the other party, by the President for the time being of the American Institute of Architects or of the American Society of Civil Engineers, and the award of such arbitrator shall be final and binding on both parties.

Such reference, except on the question of certificate, shall not be opened up until after the completion or alleged completion of the works, unless with the written consent of the Owner or Engineer and the Contractor. The arbitrator, or arbitrators, shall have power to open up, review, and revise any certificate, opinion, decision, requisition, or notice, save in regard to the said matters expressly excepted above, and to determine all matters in dispute which shall be submitted to him or them, and of which notice shall have been given as aforesaid, in the same manner as if no such certificate, opinion, decision, requisition, or notice had been given. Should either party refuse, or neglect, to supply the arbitrator or arbitrators with any papers or information considered necessary by him or them to arrive at a decision in the matters referred to him or them, which papers or information have been specifically demanded in writing, he or they are hereby authorized and empowered by both parties to the contract to proceed in *ex parte* proceedings, and such proceedings shall be final and binding.

Upon every or any such reference, the costs of and incidental to the reference and award, respectively, shall be in the discretion of the arbitrator or arbitrators, who may determine the amount thereof, or direct the same to be taxed as between attorney and client, or as between party and party, and shall direct by whom and to whom and in what manner the same shall be borne and paid.

This section caused more debate among the members of the Royal Institute of British Architects and more differences of opinion between the British Architects and Builders than all the other sections combined. The discussion on this subject, in the *Journal* of the Royal Institute of British Architects, is especially worthy of study.

The importance of this section justifies the most careful study which can be given to it. Around it would center the much discussed

authority of the Engineer or Architect to settle all matters of every kind and description. Everyone who has exercised such powers must realize that while they are theoretically desirable, their proper curtailment would unravel a very difficult situation, and would permit questions being considered in a more equitable light for all parties.

The usual American arbitration clause is a delusion and a snare to all parties. Many have felt that it was a guaranty of redress for possible wrongs, only to be awakened when its provisions were called into action.

It should be noted, of course, that the English "Arbitration Act, 1889," is a very practical document, and probably gives more legal security to the enforcement of arbitration clauses than would be possible under the laws of most of the States. Arbitration clauses are used in many contracts, and they should be made as practical and workable as possible; then, if it is found that even under the most practical of clauses the State laws need strengthening to make the term "arbitration" a reality and not a myth, there would be a good foundation on which to advocate an arbitration act similar to the English statute.

The above section leaves the Engineer as the deciding party between both Owner and Contractor, but should either party formally dissent from his decision, a notice can be given of an appeal to arbitration. Such appeals, however, will not be actually submitted to the arbitrators until the substantial completion of the work. This seemed to be the point of greatest disagreement between the British Architects and Builders. The former indorsed the above procedure, while the Builders desired the matters to be submitted to arbitrators at any time. The objection to permitting submissions at any time is that, under the broad powers which it is desirable to grant to arbitrators, it would cause confusion during construction, and make it possible to confuse hopelessly the power of the Engineer and the arbitrator. After the substantial completion of the work, such conflict cannot well arise. The Engineer being supreme during construction and after completion, the arbitrator has power to review only certain of the Engineer's decisions to which written exception has been taken, as herein provided. Of course, in the matters which are referred to arbitration, the decision of such arbitration is final. Such a policy, by its very broadness, would tend to make Owners, Engineers, and Contractors more willing

to listen and consider the grievances, real or imaginary, of the other parties.

Section XXXV is intended to dissuade both parties from taking unnecessary or trivial appeals from an Engineer's decision.

While the foundation of the arbitration section is laid on the lines of that of the Royal Institute of British Architects, it differs in many details, and its importance justifies giving the R. I. B. A. section in full, for purposes of comparison.

ARBITRATION SECTION OF ROYAL INSTITUTE OF BRITISH ARCHITECTS.

"32.—Provided always that in case any dispute or difference shall arise between the Employer or the Architect on his behalf and the Contractor, either during the progress of the works or after the determination, abandonment, or breach of the Contract, as to the construction of the Contract or as to any matter or thing arising thereunder (except as to the matters left to the sole discretion of the Architect under Clauses 4, 9, 16, and 19, and the exercise by him under Clause 18 of the right to have any work opened up), or as to the withholding by the Architect of any certificate to which the Contractor may claim to be entitled, then either party shall forthwith give to the other notice of such dispute or difference, and such dispute or difference shall be and is hereby referred to the arbitration and final decision of ....., or, in the event of his death or unwillingness or inability to act, of ....., or, in the event of his death or unwillingness or inability to act, of a person to be appointed on the request of either party by the President for the time being of the Royal Institute of British Architects, and the award of such Arbitrator shall be final and binding on the parties. Such reference, except on the question of certificate, shall not be opened until after the completion or alleged completion of the works, unless with the written consent of the Employer or Architect and the Contractor. The Arbitrator shall have power to open up, review, and revise any certificate, opinion, decision, requisition, or notice, save in regard to the said matters expressly excepted above, and to determine all matters in dispute which shall be submitted to him, and of which notice shall have been given as aforesaid, in the same manner as if no such certificate, opinion, decision, requisition, or notice had been given. Upon every or any such reference the costs of and incidental to the reference and award, respectively, shall be in the discretion of the Arbitrator, who may determine the amount thereof, or direct the same to be taxed as between solicitor and client or as between party and party, and shall direct by whom and to whom and in what manner the same shall be borne and paid. This submission shall be

deemed to be a submission to arbitration within the meaning of the Arbitration Act, 1889."

XXXV.—Damages for Submission to Arbitration.—In submitting to arbitration questions in dispute, or differences involving a variation in the amount of the contract, both the Owner and the Contractor shall have the option of submitting their appeals in separate items or groups of items. In which case the arbitrators or arbitrator, in awarding a judgment on each item or group of items, are authorized and directed by both parties to the contract, to award to the party whose contention is sustained, in addition to the sum actually determined as correct, an amount equal to .....% of such amount. Such additional sum shall be added to or deducted from the amounts in question as the case may require. Should a claim be set aside in whole, or should the nature of the case otherwise require it, the amount of such additional award shall be added to or deducted from the amount due the Contractor on the contract. These additional sums shall be considered as damages for the time and trouble incident to such appeal.

The object of this section is to act as a check to prevent appeals to arbitration becoming the fashion rather than the exception. While the principle of this section is considered good, the ability to enforce it is not settled in the writer's mind.

General Conditions.

The endeavor has been to make the Agreement and Schedule of Conditions of Contract as broad and general as is consistent with good practice. There are additional conditions which may be desirable or essential for particular buildings, but as these conditions are special and of only limited, if not personal, application, it is unwise to attempt to write such in a general set of Conditions of Contract.

Legally, such conditions would be just as binding upon a Contractor whether they were given as a part of the Agreement or were placed in the Specifications. The writer, therefore, advocates placing in the fore part of a Specification a chapter headed "General Conditions," under which would be placed special requirements of a more or less legal and general nature, which, for the purpose of the particular building, might have been made a part of the Schedule of Conditions of Contract.

The following are given as samples of clauses which might be found under a heading "General Conditions." It must be distinctly

understood that these clauses are given only as samples, without comment. The writer does not endorse them for general use, and, except in isolated cases, he considers that many of the provisions might be more harmful than beneficial.

Checking Documents.—The Contractor shall carefully study and compare all drawings, specifications, and other information given to him by the Engineer, both as to figures, materials, and methods of construction, and shall report in writing to the Engineer for rectification any error, inconsistency, or omission therein, which he shall discover, or any modification he considers necessary or desirable for the stability or economical construction of the building.

Doing Work in Freezing Weather.—Work shall not be done when the temperature where work is being done is below 28 degrees Fahr., or if exposed to rains, except by special permission of the Engineer. Frozen materials shall not be built upon.

Engineer's Approval.—All materials or work specified to be of an "approved" type, construction, or finish, shall be submitted to and approved by the Engineer before being used or executed. Where materials, quality, or finish are not distinctly specified, samples shall be submitted to and approved by the Engineer before using same. An approval by the Engineer shall be understood to mean that it is conditional upon the work being in conformity with the contract requirements. No approval of deviation from contract requirements shall be implied, but shall be specifically mentioned in the Engineer's written approval. (See Section IV, Schedule of Conditions of Contract.)

Materials and Workmanship.—All materials shall be new, and all materials and workmanship shall be the best of the respective kinds contemplated by the contract, unless otherwise specified.

Contemplated Work.—The work to be executed under the contract consists in general of a ....., constructed and completely finished in every particular ready for furnishing, within the limits shown on the Drawings and described in the Specifications, except (*a*) elevators, (*b*) lighting fixtures, (*c*) electrical, (*d*) heating work, (*e*) or work specifically excepted or made subject for separate bids.

Contract Documents.—(See Agreement, Article I.)

Definitions.—The following definitions of works, terms, etc., shall be understood to govern their use throughout the Contract:

> *Approved.*—The word "approved," as applied to any material, manner or mode of construction, shall be construed as though followed by the words "by the Engineer."
>
> *Best.*—The term "best," or "best quality," shall be taken to mean that there is no better quality or finish of material of the

grade specified or reasonably implied, obtainable on the market supplying the vicinity, or that there is no better class of work mentioned, reasonably obtainable.

Net Cost.—(See Conditions of Contract, Section XXVII.)

Provide, Etc.—For the sake of brevity, and to avoid needless repetition, the imperative mood has been largely used throughout the specification. Wherever it is used it shall be understood to mean, "The Contractor shall furnish all material and perform all labor necessary to .....," and these words shall be read into the sentence.

Provisional Amount.—(See Conditions of Contract, Section XXVIII.)

Drawings.—The Contractor shall furnish to the Engineer at proper times all shop and setting drawings or diagrams which the Engineer shall deem necessary to carry out the work. Each drawing or diagram shall give all the information required by the nature of work which it is intended to govern. These drawings or diagrams shall conform strictly to the Engineer's drawings and specifications, and where possible upon measurements taken from existing work. No deviation from the Engineer's drawings and specifications shall be made unless by special written order. The Engineer will inspect such drawings, but his purpose being to aid the Contractor and expedite the work, he shall not be held responsible for the Contractor's errors, the result of which, should they occur, must be made good at the Contractor's expense. The Contractor shall make required changes in such drawings or diagrams and supply the Engineer with one correct copy. Any models or templates submitted shall be changed as reasonably required until satisfactory.

Fires, Smoking, Etc.—The Contractor shall not permit open fires to be lighted at any time in or dangerously near the building, and, after the roof is on, shall prohibit smoking within the building.

Imperfect Construction.—The Contractor shall notify the Engineer in writing if at any time he finds that any work or material that has or is being furnished by another Contractor in connection with his work is imperfect; and in case of his failure to give such notice, the Contractor shall be held to have accepted such work or materials, and all other existing conditions, and shall be responsible for any defects in his own work consequent thereon, or any change to his work made necessary by the future correcting of such defective work by other Contractors, and shall not be relieved of the obligation of any warranty because of any such condition or imperfection.

Imperfect Design.—The Contractor shall notify the Engineer in writing if he thinks any part of the work as designed or constructed is unsafe or likely to become so, or unsuited to produce the intended

results, and shall thereafter proceed with such work only on the order from the Engineer.

Lines and Levels.—The Contractor shall establish all lines and levels necessary to the proper locating, laying out, and erecting of the work, in accordance with the Owner's survey and Engineer's drawings, and shall be responsible for their accuracy and maintenance. (See Schedule of Conditions of Contract, Section V.)

After the foundations have been completed and before the first tier of beams is set, the Contractor shall file with the Engineer a certificate from an approved surveyor showing that the building is located correctly according to the Owner's survey, and that the foundations, column bases, etc., are correct according to the drawings.

Measuring Work in Place.—The Contractor shall measure work already in place to insure the proper execution of subsequent work, or to assist the Engineer in making his drawings, and should any discrepancy between the executed work and the drawings be discovered, he shall report at once such discrepancy to the Engineer.

Notifying Engineer.—The Contractor shall bring to the attention of the Engineer any conditions pertaining to the site, the soil, the various portions of the work, whether executed by himself or by other Contractors, or to proposed work, or to laws and ordinances governing the work that should be known by the Engineer.

Overloading Structure.—The Contractor shall not permit any part of the structure to be loaded during construction with a weight greater than it is calculated to bear.

Reports to Engineer.—The Contractor shall furnish reports to the Engineer showing the status of the work at the building and in the shops, whenever so required, and in the manner directed.

Signs.—The Contractor shall not permit signs upon the building or premises, other than that of the General Contractor, except by the written permission of the Engineer.

Storage Limits.—The Contractor shall confine the storage of materials and the operations of his workmen to the reasonable limits indicated by the Owner or Engineer, and shall not unnecessarily encumber the premises with his materials.

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PAPERS AND DISCUSSIONS

This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

THE EFFECT OF ALKALI ON CONCRETE.

By George Gray Anderson, M. Am. Soc. C. E.

To be Presented February 16th, 1910.

To some extent, this paper is a continuation of the informal discussion on "Irrigation" which took place at the Annual Convention of the Society, held in Denver, Colo., in June, 1908.*

In that discussion some reference was made to the introduction of more permanent structures in irrigation work; that, for such, concrete was being used extensively; and that, in some smaller structures, failures had occurred, in the form of crumbling or disintegration, due, presumably, to the action of the alkali frequently present both in the soil and in the water, and resulting from chemical reactions between the sulphates in these waters and the cement, which reactions may not yet be fully understood and are under investigation. Since that time the subject has received more than passing attention from the Profession and the industry at large, as shown by more or less extended comments in the technical and trade journals.

Several instances of such failures were referred to by members who took part in that discussion, and it is largely the purpose of this paper to consider these and other instances somewhat more fully, to elicit further discussion and consideration, and, if possible, to arrive at more

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

**Transactions*, Am. Soc. C. E., Vol. LXII, p. 1.

definite conclusions in reference to the disintegrating action of alkali, in the soil and in the water, and of the methods of counteracting it.

It should be stated, at the very outset, that the subject is not approached with alarm, but rather with an inquiring mind.

Irrigation development has passed the stage in which temporary structures are considered; and the Engineer is looking for permanence, from the economic standpoint of reduction in cost of maintenance, operation, and renewals. He is almost ready to consider favorably the use of concrete, in all forms and for nearly all structures. Under observation, certain defects, arising from natural conditions which are almost always present in the regions of operation, have become apparent; and these cause a temporary hesitation and further consideration of plans and methods. The attitude is by no means that of abandonment, but rather of painstaking investigation of all the conditions, with the desire to ascertain the cause, and, from that, to introduce methods which will overcome the defect. Most certainly it is not the desire to magnify unduly the significance of a few isolated instances of failure, or to ignore defective constructional methods in these, but rather to investigate methodically all phases of the failed structures. Such investigation is now being made by the chemical branch of the Reclamation Service, as well as by others, and until official reports are published, such discussions as the present may be considered premature. In view, however, of the great amount of work under way and in contemplation in regions known to be affected, and the serious importance of these matters, not only to irrigation engineers, but to all members of the Profession engaged in structural work using concrete in contact with alkali (in soil or in water), it is not out of place to add the available knowledge and information to the more extended and methodical inquiries.

It is well known, of course, that alkali is present in greater or less amount in the soil of the arid regions. In the disintegration of the rocks, the minerals have been broken down into mineral salts which have remained in the soil as soluble salts. The rainfall and the drainage of the region are insufficient to carry these salts out of the soil through the subsoils and eventually into the streams and rivers, as is the case in the more humid regions.

The accumulation explains the wonderful fertility of the soil in arid regions, and at the same time accounts for the menace to the purely

agricultural interests. In sandy soils, with free drainage, irrigation assists in the removal of these salts; in the closer soils, such as shale, with less drainage, it may and does increase the accumulation to a harmful degree.

All are familiar with the results in connection with agricultural operations. Efflorescence in large amounts is a sure indication of alkali, and then it is in its least harmful form. What is known as "black alkali" is much more injurious to plant life. The latter form represents the alkaline carbonates, which actually dissolve the organic materials of the soil, and kill the germinating seeds or roots of the plants; the former, the white alkalis (of which the most common are sodium sulphate, sodium chloride, magnesium sulphate, and magnesium chloride), are not in themselves poisonous to plants, nor do they attack the substance of the plant roots; but they are injurious when in excessive amounts, as they prevent the plants from taking up their needed food and water supply.

It has been tolerably well determined just what amount of soluble salts is limiting for plant growth, varying with the character of the salts, of the soils, and of the plant itself; for crops differ in their ability, and some will thrive on soils on which others would languish.

Such, very briefly, is the general action of "alkali" on agricultural interests. The salts are largely the sulphates of lime, magnesia, soda, alumina, iron, and potash, and the carbonates of soda and potash. Of these, the sulphates only may prove injurious to concrete, and as the shales of the Dakota formation are known to be rich in alkalis, and are common throughout the arid regions, the effect of these soils and their salts on concrete structures will be set out by brief reference to a few known cases where this action has taken place.

A good many, if not all, of the projects of the Reclamation Service are located in alkali regions, and, thus far, the attention of the engineers has been directed to the failure of concrete structures on only two of these projects—one in Montana and one in Wyoming.

These failures may indicate some special peculiarity of the local soil, and may be somewhat similar to the well-known fact that the action of sea water has been destructive to concrete in certain localities and not in others.

The Service has started an investigation, which is still continuing, and the chemist's report has not yet been published, but Mr. Jewett,

the cement expert of the Service, in a paper before the American Society for Testing Materials, set forth some of the conditions, as will be shown by the following extracts*:

"On the Sun River project * * * its effect has been seen in the disintegration of a number of small structures, such as pipe culverts, which are partially submerged in the small streams carrying the drainage of the surrounding country. The effect of this water in causing the disintegration of the concrete is noticed, both in the softening of the material where constantly submerged for a period of three or four months, and also more particularly in the breaking up of the structure at the line where the water rises and falls, or, in other words, where there is a submersion followed by a drying out, followed by another submersion, etc. This process brings about a breaking up of the structure, similar in appearance to that of the effect of freezing, only much more severe. It is described by the engineer in charge of this work as follows:

" 'On examination of some of the concrete culverts which have been in place in alkali water for three or four months, the part under water is much softer than the part above water, but the greatest effect of the alkali seems to be at the water surface. Have examined pieces of these culverts taken from below water level and found, by microscopic examination, minute crystals which disintegrate on being exposed to the air and become a fine powder. This process of crystallization has an enormous expansive force, the same as freezing, and this undoubtedly has an enormous disintegrating effect. One culvert which had been in alkali water several months had no strength at all and was like so much sand and mud at the water level.'

* * * * * * *

"In studying the cause of this disintegration it seemed that a chemical analysis of the water producing these results would be the first step of importance to be taken. The results obtained from an analysis of this water are as follows:

	"Milligrams per Liter.
"Calcium sulphate ($Ca\ SO_4$)	1 690
Magnesium sulphate ($Mg\ SO_4$)	6 870
Basic magnesium carbonate ($MgH_2\ CO_2$)	305
Magnesium chloride ($Mg\ Cl_2$)	192
Potassium chloride ($K\ Cl$)	20
Total solids	9 077
Weight after ignition	8 855
Loss on ignition	222

* *Proceedings*, Am. Soc. for Testing Materials, Vol. VIII, 1908.

"From this analysis it will be seen that the principal mineral ingredient of this water is magnesium sulphate, and this leads to the conclusion that the effect of this water in causing disintegration of concrete is similar in its action to that of sea water in which magnesium sulphate is stated to be the principal substance acting to bring about the disintegration of concrete with which it comes in contact."

Professor Headden, of the Colorado State Agricultural College, has at various times issued bulletins in reference to alkali in the soil and in the water, and has recently added to his publications Bulletin 132 on "The Destruction of Concrete by Alkali," in the course of which he gives an account of his investigation of cement tiling which had rapidly deteriorated. From that bulletin the following is quoted in order to convey clearly the conditions.

"The following case of deterioration may be of some interest as it presents some new points. There were sent to me some months ago a number of fragments of tiling which had been laid but from eight to nine months. They had been disintegrated to such an extent that one of the samples was simply a white putty-like mass mixed with sand. There was nothing about this sample remotely suggestive of concrete. Another of the samples consisted of a fragment of the tile, the interior portions of which had been wholly decomposed, while there still remained an outer and inner portion in good condition, or comparatively so. Another sample had been attacked in the inner side leaving the mass of the tile outside of the zone of decomposition apparently sound. The line of decomposition was sharply limited and showed distinctly, but even in this case there remained a thin layer of cement on the inside.

"I do not know whether the tile were running full of water or not, probably not, as the fragments seem to be parts of 16 or 18-inch tile. The importance of this point is simply this, that it would answer any question in regard to the separation of solid 'alkali' salts on that portion of the tile above the water line. That such a separation of these salts should take place does not seem very probable in this case.

"The point of attack, so far as the samples at my disposal indicate, is either in the center of the cement mass or near the inner surface. In either case we are almost compelled to assume the action of the water, *i.e.*, of the salts held in solution. At the same time we see the resisting power of the outside and inside surfaces, which are evidently richer in cement than the inner portion of the mass of the tile.

"The sand used in making the tile was quite fine. The proportion used was 5:1. We can readily see that the individual masses of cement in the interior mass of the tile must be small and readily attacked by solutions of salts capable of reacting with the cement.

"It seems quite evident that the agents causing this disintegration of the cement must gain access to the interior portion of the mass of the tile in the form of a solution, but it is a question whether this solution is the same in strength and character as the water in the drain. I think that I have clearly shown in Bulletin 65 of the Colorado Experiment Station, page 34, that the water soluble portion of the soil is different from the salts held in solution in the ground water. Further, in Bulletin 72, page 28 *et seq.*, that the ground waters are different from the drain waters, and the action of these waters on the tile would vary somewhat according to the source of the solution which penetrated the mass of the tile.

"The drain waters obtained, say at a depth of four feet, contain as a rule a smaller amount of salts in solution than the ground water. The salts present are the same, but their relative quantities vary. The usual salts present in the ground waters are, sodic sulfate, calcic sulfate, magnesic sulfate, sodic carbonate, and sodic chlorid, in some instances also magnesic chlorid. In the drain waters we have the same salts, but their relative quantities are different, the most notable feature being the relatively large amount of sodic carbonate.

"The substances claimed to be the most active in effecting the disintegration of cement are the sulfates. In the case of sea water the explanation offered is that the lime of the cement decomposes the magnesic salts present in the water, whereby the lime goes into solution and the magnesia is deposited in its stead, causing a bulging and disintegration of the work owing to its greater bulk. The principal magnesian salt in sea water is the chlorid, the deleterious action of which on cement is not established, but it has been shown that magnesic sulfate acts energetically. The most active agent in decomposing concrete is the sulfuric acid of the sulfates carried in solution. Again, solutions of gypsum have been shown to act detrimentally on concrete by forming with the tri-calcic aluminate a sulfo-aluminate.

"The water acting on these tiles, whether it be ground water or drain water, is very bad. One of the two small samples sent to me contained 1 252.6 grains per imperial gallon, of which 493.5 grains were sulfuric acid (SO_3) with an extremely large proportion of magnesia. The other contained 542.3 grains per imperial gallon, with 42.31 grains of magnesic oxid or 7.8 per cent. of the total solids. I have examined a number of waters from this section, and find the ground water uniformly heavily laden with the sulfates of lime and soda, with comparatively large quantities of magnesia, especially if the water comes from near the surface, as it necessarily must in the case of drain waters. In addition to the above constituents, there is uniformly some carbonate present, which for the present purpose we may consider as sodic carbonate.

"We have then an ample supply of these substances present which

are known to act deleteriously upon cement, *i. e.*, the sulfates, a portion of which is magnesic sulfate, and in addition there is so much sodic carbonate present that its action cannot be neglected."

Professor Headden, assuming that the cement used in the tiling was an average Portland cement, with not more than 2% of calcic sulphate, then considers the action of the waters on the tile.

The white decomposition product indicated the presence of a large quantity of water, also of considerable quantities of sulphuric and carbonic acids.

The decomposed mass was washed with distilled water, and the wash-water showed that it had taken sulphuric acid, lime, and soda into solution.

An analysis of the Arkansas River water at Rocky Ford, which, Professor Headden remarks, is essentially a return water, is given, and the assumption is made that the composition of the water acting on the decomposed tiling was much the same, as are all waters in that district, varying in amounts of mineral matter, or total solids held in solution, as shown by the two samples of drain water accompanying the tiling—one having 1 252.6 grains, the other 542.3 grains.

The action on the tiling seems to be similar in Colorado and Montana. A comparison of the features of construction may not be possible.

In the Montana case, Mr. Jewett says:

"Of course, in this case, as in any other case of concrete failure, the question of poor workmanship is one of the first to be considered and it should be noted that the work in question was carried on by force account under direct supervision of employees of the Service and not by contract, and was done in the usual thorough and substantial manner, in which all of the construction work of the Service is carried on. After taking all the evidence into account there seemed to be no other conclusion but that the action of the alkali water was directly responsible for the results noticed."

Professor Headden has not much information on the quality of the tiling examined by him. The sand used was quite fine, the proportion 5 to 1, and the individual masses of cement in the interior must have been small and readily attacked by solutions of salts capable of reacting with the cement.

Both Mr. Jewett and Professor Headden find the decomposing action, in the cases of these tilings, to be similar to that of sea water.

in which magnesic sulphate is stated to be the principal agent. Professor Headden says, as already quoted,

"The principal magnesian salt in sea water is the chlorid, the deleterious action of which on cement is not established, but it has been shown that magnesic sulfate acts energetically."

Reference to this similarity to the action of sea water will be made later; meantime, the following is a summary of Professor Headden's conclusions.

The analysis may be interpreted as indicating that the cement had been deprived of silicic acid and of a relatively large amount of lime. The amounts of the ferric and aluminic oxides have been reduced to about one-half as much in the decomposed cement as in the original. The sodic sulphate, more particularly the calcic and magnesic sulphates which are present in the ground-water, will at the same time act on the tri-calcic aluminate, forming the sulpho-aluminate, which crystallizes with 60 molecules of water and is soluble. The increase of sulphuric acid (more than 20%), and also of carbonic acid (more than 12%), in the water-free decomposition product is noted, as compared with only about 1% each in the original product, while magnesic sulphate has also contributed to the result, being more than four times greater in the decomposed product than in the original; and there is the conclusion that:

"The action of the ground waters seems to have been one of chemical change, converting the lime of the cement into a sulfate and carbonate accompanied by the removal of silicic acid, alumina, and lime, and totally destroying the cohesiveness of the concrete."

The case of the sewers at Great Falls, Mont., has been referred to frequently, and is probably the most striking illustration of deterioration yet considered. The City Engineer, Mr. C. W. Swearingen, was so impressed by the conditions that he requested the authorities at the Montana State Agricultural College to investigate and report on it.

The preliminary conclusions of Mr. E. T. Tannatt, Rural Engineer, and Mr. Edmund Burke, Chemist, are embodied in Bulletin No. 69 of the Montana Experiment Station, from which the following extracts give a very clear exposition of the conditions:

"The Third Street main sewer was constructed in the year 1890, the material used being Portland cement pipe of home manufacture, made in moulds and put in place after the cement had hardened.

Sewer is oval in shape, dimensions 26 x 32 inches. This sewer, after a comparatively short term of service, showed defects of so serious a character as to necessitate rebuilding portions of the same. The remaining portion of this sewer has disintegrated in many places and occasionally portions of the pipe are gone, the cement in places has the appearance and consistency of fresh lime mortar.

"The Sixth Street main was constructed in the year 1892, the material being the best grade of common brick of home manufacture. Sewer is circular in form, four feet inside diameter, two rings of brick laid on edge in imported Portland cement mortar, outside plaster coat one-half inch thick, not plastered on inside; all cement mortar one part cement and three parts sand. The masonry in the invert in this sewer is disintegrating throughout its entire length, the defects being confined almost entirely to the invert and lower sections of the manholes; the arch is in apparent good condition.

"There is no sub-drain under, nor on the sides of the sewer. As stated before, the disintegration in the Third Street main, where cement pipe was used, commenced soon after its construction, whereas the Sixth Street main gave no evidences of defect for about seven years after its construction, and it is only during the past four years that decomposition has been rapid."

The examiners found: (*a*) that the disintegration and destructive action of the cement did not come from adulteration, nor from the quality of the materials used; (*b*) that though a limited chemical or physical action of the alkali salts may be a partial cause of the breaking down of the brick, yet the principal cause was the fact that the bricks were too soft, and not fit for sewer construction; and (*c*) that the action on the cement was chemical, that the destructive agent came from the outside of the sewer, and that the disintegration of the cement cannot be charged to any action of the sewage or gases from the same.

They found that certain portions of the sewers showed the presence of external moisture to a more marked degree than other sections, and that, contiguous to it, were various depressions in which water collected and gradually seeped into the soil, thereby affording means for the leaching out of the alkali from the gumbo, through the sand and other more permeable formations, into the back-filling adjoining the sewer, there depositing against the structure. The soil samples showing the highest percentages of alkali were taken immediately at the back of the cylinder of the sewer.

Soil samples were analyzed in order to ascertain the amount of alkali salts, whether these were being leached out and lodged against

the sewer, and the amounts at various depths, but there is no chemical investigation of the action on the cement and its alteration, which it is understood will form the subject of a later bulletin.

The analyses embodied in the report, however, show that the maximum disintegration of the sewer occurred where the greatest percentage of alkali showed in the soil, and that the ground-waters were leaching the alkali from the soils and depositing the salts along the sewer.

Messrs. Tannatt and Burke consider that the use of cement for sewers and other structures in alkali soil is inadvisable in cases where it is not possible to drain all ground-waters from the exterior of the sewer, and they recommend that drain tile be placed in the bottom of the trench, covered with clean gravel or crushed stone, and that similar drains be placed around the barrel of the sewer to provide effective drainage of the ground-waters, in order that it will not be possible for them to come into contact with the structure itself, and, where these precautions cannot be taken, they "unhesitatingly recommend that cement pipe be not used."

The writer had some experience with a construction somewhat similar to the Great Falls sewer, and in similar, though not such aggravated, conditions, but with entirely different results.

Lake Loveland was constructed under the writer's supervision in 1894. The outlet is a tunnel about ¾ mile long, nearly all of which is in stratified shale. The rock was of such a character that it would have decomposed under the action of passing waters at a high velocity, and therefore the tunnel was lined. The interior diameter of the tube was 5 ft. The invert was built of two rings of well-burned brick, and the arch was of concrete.

In the excavation, large volumes of water were developed, ultimately giving a flow of about 2 cu. ft. per sec. The water, undoubtedly, was impregnated heavily with alkali, but the writer did not obtain a chemical analysis of it, as, at that time, he did not suspect that any deleterious action might be caused by the character of the water itself; but, apart from the fact that it was "return water"—the drainage of a large section of irrigated country lying west of the reservoir—there were constant physical evidences that it contained salts, if one might judge from the action on the human system caused by drinking it in moderate quantities.

Through the whole length of the tunnel a sub-drain was channeled out in the rock bottom, and wooden boxes were placed to convey these waters. Side-drains were also placed at frequent intervals throughout the whole length, and mostly where the volume was largest, always connected with the under-drains. These were carried up as high as the drainage showed, usually to the spring of the arch. In short, every precaution was taken to prevent the water from coming in contact with the concrete, certainly until it had set thoroughly, and solely for the purpose of securing good results and without thought of any ultimate chemical reaction.

The work was done practically by force account. The materials were supplied, the labor was done under contract, and the work was in charge of a competent concrete worker and under a thoroughly able inspector.

As far as the writer knows, the concrete and the cement mortar in the brickwork are in thoroughly good condition to-day. It is nearly 8 years since he examined the work, and then it was found to be without any apparent defect. The superintendent informed the writer, recently, that he had not been in the tunnel for 2 years, but had then found it in perfect condition, and was of the opinion that it still is so.

One other structure may be mentioned, a reservoir spillway in the Arkansas Valley, or the Bob Creek Reservoir, constructed and owned by Mr. W. C. Bradbury. The writer's attention was attracted to it in September, 1909, when passing near it, on other business. He first noticed and examined a small division head-gate about ½ mile below the reservoir. It was stated, that these works had been built less than 2 years before that time. The writer had no means of knowing the proportions used in the work, but this head-gate seemed to have been fairly well built, and its upper portion was in fair condition. The floor had evidently been about 6 in. in thickness, but it was entirely decomposed, and could be crushed readily in the hand. The uprights, up to about the normal height of the water, showed disintegration. At the outlet from the reservoir, the apron was disintegrated in the same way. It had been reinforced, and the reinforcing material was exposed and bright while the concrete mass was almost completely pulverized.

The spillway to the reservoir was of considerable length and height, of ogee overfall, and had the same appearance of fairly good construction. Prior to the writer's visit, there had never been any water flow-

ing over it from the reservoir. Its lower portion, for a height of about 10 ft. above the apron level, was markedly discolored, and on breaking the surface crust, it was found that the interior mass of the concrete could easily be removed and pulverized in the hand.

About a month later, water passed over the spillway and washed out the lower section of the fall to the height already indicated. The concrete had been placed on a foundation of shale, from which the alkali water oozed. No provision had been made for the under-drainage of this water, and the conclusion is inevitable, that the destruction of the concrete was the result of the chemical action of the alkali in the water draining from the shale.

From these various instances, it would seem that somewhat new problems are presented to the constructing engineer, or, if they are not new, they are old problems under new conditions, and engineers have been prone to overlook their significance until serious consequences have forced them directly on their attention.

It is very likely true that the effects of alkali on concrete are similar to those of sea water, with some differences, as these in themselves differ in various localities. Means of overcoming the sea-water action are fairly well outlined, though as competent an authority as Feret says:

"No cement or other hydraulic product has yet been found which presents absolute security against the decomposing action of sea water."

Michaelis, on the other hand, long ago made certain recommendations which may be summarized briefly here: From the physical point of view, a dense concrete surrounded with an impermeable layer composed of 1 part of cement and 2 or 2½ parts of sand of mixed grains, of which at least one-third must be very fine, and the whole mass made, if possible, in one operation; from the chemical point of view, the use of cement rich in silica and as poor as possible in alumina and ferric oxide, and the use of substances which render the mortar, at any rate in its external layers, denser and more capable of resistance.

The solution of the problem, from the chemical point of view, will be the care of the chemists and the cement manufacturers. The problem for them is as new—or as old under new conditions—as it is for the engineer; and, conceding the general proposition that the solution lies in good construction, which the engineer must realize and will provide, there is, it would seem, the larger problem of a coating which,

adding impermeability, shall also resist the chemical reactions between the soil and water and the cement.

What is good construction under these conditions?

It seems to the writer that the engineer, under these conditions, first has to extend materially the field of his investigations before outlining plans. The character of the soil, and of the water passing over the soil and to be transported over the structure, must be thoroughly analyzed. Then, to the physical problem, similar analytic consideration must be taken of the materials used as concrete aggregates—the sand, the gravel, and the water.

On that feature, the following is quoted from Mr. Jewett's paper:

"In the case of the sand used on the Reclamation Service project above mentioned, the engineers reported that there were evidences that this sand, which was the only supply readily available, contained alkali or some substance making it chemically active and assisting in the disintegration of the concrete. A chemical analysis of this sand was therefore obtained and resulted as follows:

	Per Cent.
"Silica ($Si O_2$)	67.03
Alumina ($Al_2 O_3$)	6.85
Ferric oxide ($Fe_2 O_3$)	2.50
Manganese oxide ($Mn O$)	.02
Lime ($Ca O$)	10.72
Magnesia ($Mg O$)	1.72
Sulphuric anhydride (SO_3)	Trace.
Alkalies { Soda ($Na_2 O$)	1.74
Alkalies { Potassa ($K_2 O$)	1.51
Water at 100° C.	.14
Ignition loss	8.16
Total	100.39

"This sand gives excellent results from physical tests but since, as will be seen from the above analysis, the alkali content is about 3 per cent. which would correspond to about 15 to 20 per cent. of feldspathic material and that it also contains a considerable percentage of limestone, it is altogether possible that this sand would be more readily acted on by the influences tending to disintegrate the concrete than a sand of a more highly silicious nature."

It is extremely likely that similar conditions will be found in most of the sands and gravels available in the arid regions where alkali is known to exist.

There is no occasion for elaboration on the care required in the selection of water for use in mixing concrete. It is quite well known that the ground- and drain-waters in most of these districts are largely impregnated with alkali; that is discernible without chemical analysis, and is particularly true in the lower reaches of irrigated valleys, where most of the waters are drain-waters, consisting of the concentrated essence of alkali from the irrigation and drainage of the upper valleys. Prior to the introduction of the gravity system at Greeley, for instance, the domestic supply from underground galleries contained as much as 1 800 parts of hardness, and, it may be added, the water used in mixing the concrete at the Loveland work referred to, was obtained, not from the drainage of the tunnel, but from the town supply withdrawn from the river inside the foot-hills.

If the action of the alkali salts in the water surrounding a concrete mass can have the destructive effect shown in the various instances quoted, it is certain that the incorporation of such water in the aggregate cannot be other than injurious.

Beyond extra care in the materials, and the production of a dense mass, there is no room to doubt the advisability of effective drainage around the structure.

Professor Headden, in Bulletin 72,* speaking of the alkali lands in the Poudre Valley, said:

"I am convinced that the only question of alkali that we have resolves itself into one of drainage, and beyond this, there is no alkali question for us."

It would be too much to hope that the drainage around structures would as effectively solve the difficulty of alkali in contact with concrete, but it will aid in resisting the action of the agencies at operation.

It is not possible, however, in all cases to provide such drainage as would be effective, and beyond all that—beyond the external application and the possible incorporation of an impervious coating in the mass itself—it seems to the writer that there must be added to it or in it, in some form, agencies which will counteract the effect of these apparently well recognized chemical elements which bring destruction; and that must be left to the chemist, on whose province the writer has probably encroached too far, and, apparently, with only a speaking acquaintance with that science.

* Montana Agricultural Experiment Station.

It has been the writer's purpose to bring the subject to the attention of the members, with, perhaps, the selfish motive of eliciting discussion and enlightenment on a subject on which he is looking for light and guidance, in a continued consideration of what seems to be one of the most important subjects than can secure the attention of Western engineers, at this time. The writer is aware that there are others who are far more competent to treat this subject learnedly and practically, and will be content if his efforts have been sufficient to draw from them the knowledge and enlightenment which they possess.

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PAPERS AND DISCUSSIONS

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WATER SUPPLY FOR THE LOCK CANAL AT PANAMA.

Discussion.*

BY MESSRS. C. E. GRUNSKY, H. F. HODGES, THEODORE PASCHKE, AND ALLEN HAZEN.

Mr. Grunsky. C. E. GRUNSKY, M. AM. SOC. C. E.—In an article on the type of the Panama Canal,† prepared by the speaker in January, 1909, attention was called to the water requirements of the Canal, as estimated by the Board of Consulting Engineers of 1905, on the basis of figures furnished by General Henry L. Abbot, of the Corps of Engineers, U. S. Army (Retired). At that time it was stated that these figures required revision because, after the estimates had been made, the actual area of Gatun Lake was found to be 164 sq. miles instead of 110, as assumed, and because, in the meantime, it had been determined to widen the canal locks to 110 ft. Further discussion of this important feature of the Canal problem by the Canal authorities has not come to the speaker's notice.

Mr. Sorzano's paper is timely because, although it is now too late for a material change of canal plan, it would be unfortunate if proper measures were not taken to bring the lock canal to the highest state of efficiency.

The speaker is among those who were disappointed when the report of the Board of Consulting Engineers—eight to five—was set aside, and the lock canal was adopted. Locks are not added to canals to improve them. They are, however, oftentimes necessary in order to adapt a canal to the physical peculiarities of the available canal route. Where

*This discussion (of the paper by Julio F. Sorzano, M. Am. Soc. C. E., printed in *Proceedings* for October, 1909, and presented at the meeting of November 17th, 1909), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† *Popular Science Monthly*, May, 1909.

Mr. Grunsky. there is a choice between an adequate canal without locks and one with locks, the question to be answered is whether the better canal is worth the additional cost in time and money. In the case of the Panama Canal this question has never been answered conclusively. The Board of Engineers of 1905, with the data then available, told President Roosevelt that the additional cost of a sea-level canal was justified. The Canal Commissioners, who, in the light of more complete data, might have taken up the question again as late as 1908, seem to have been content to place before Congress and the American people evidence that the Gatun Dam can be made a safe structure, but they failed to furnish convincing evidence that the better canal would not have been worth the additional cost. Moreover, in the light of the favorable progress reports relating to the excavation of the canal prism, it is reasonable to believe that the excess cost would not have been as great proportionately as was originally assumed.

When the American engineer is asked why the United States did not build a first-class canal, he will be at a loss for a satisfactory answer. Nor is it likely that conclusive evidence on this point will ever be furnished by anyone. The time has passed when it was essential to study the larger problem, and, in any discussion, under comparison with the accomplished results, the sea-level canal, with difficulties of conjectural magnitude, would be likely to fare badly.

It seems to be taken for granted that a dam at Alhajuela would safeguard the water supply. The speaker has been at the Alhajuela dam site, and was impressed by the lack of adequate information relating to the geological formations at that point. Exhaustive studies may have been made since that time. At any rate, this word of caution against accepting the Alhajuela site as the best, without full investigation, will not be amiss. At Gamboa, where it would have seemed natural and reasonable to construct a dam, with spoil from Culebra Cut, to an elevation of about 200 ft. above sea level, there is good conglomerate foundation, the lowest point of which is about at sea level. This was satisfactorily demonstrated by borings in 1904. It may still be possible to construct a dam at this point. The Gamboa Reservoir, with a water surface of more than 30 sq. miles, would be a most excellent regulator of the flow of the Chagres River. If this dam had been constructed at once (1905-1907), it would have solved numerous water-supply problems, and would have furnished power for many uses along the line of the Canal. Perhaps a dam can be constructed at this point after the lock canal is in service, but this does not seem likely, because the dam site will be submerged by the waters of Gatun Lake to a depth of about 40 ft. If the work already done has not made this dam impracticable—construction with spoil from Culebra Cut being assumed—the advisability of constructing it at once should receive more than passing consideration by the Canal authorities. If, upon a revision of

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the author's figures, it be found that a shortage of water is to be feared, a decision should at once be reached with regard to the location of the additional storage reservoir. This decision should not be postponed until what, at present, seems to be the best site has been rendered impracticable.

As one of the engineer members of the Isthmian Canal Commission of 1904-1905, the speaker had hoped to have an opportunity of studying and passing upon all the larger engineering problems which the construction of the Canal involves, but this hope was not realized, and since his brief connection with the work, time and opportunity for a comprehensive study of these problems could not be found. Lacking complete information, yet being frequently credited with its possession, it is always with some misgivings that he participates in the discussion of the Canal problems. He wishes to avow himself deeply interested in the success of the Canal, though venturing some criticism.

The water-supply estimates presented by the author are demonstrable in part only. The quantity of water required for lockages, the leakage at structures, the water required for power, the rainfall, and the evaporation from the water surface of the lake can be estimated with some degree of precision from known facts; but the loss of water from wet marginal lands and the loss by infiltration from a lake of large extent are matters of conjecture, in which there is wide latitude for personal opinion. Would it not be better, therefore, to estimate the available supply—the known flow of the streams, the water available from storage, and the rain on the lake's surface—and to deduct from this the losses by evaporation from the lake, leakage at the structures, and a definite allowance for power, and see what remains? The remainder will be the water available for lockages and for the indeterminate losses by marginal evaporation, by seepage, or otherwise. These losses may turn out to be small. Perhaps it will be safe to assume no loss by evaporation from marginal wet areas, because this loss is offset by the gain resulting to the flow of the streams by the elimination of the swamps and wet areas at present within the lake contour above Gatun, for which the author has made no allowance. The loss of water by infiltration and all other indeterminate losses can then be introduced into the calculation as personal judgment may dictate.

The Canal capacity, as determined by the water supply, on the extreme assumption that such indeterminate loss is too small to be considered, cannot be exceeded except by drawing more water from storage. A draft of more than 5 ft. on Gatun Lake, as pointed out by Mr. Sorzano, would reduce the effective depth of the Canal, unless storage be provided elsewhere than in Gatun Lake.

Using for illustration such data—not verified—as are at hand, it would appear that:

Mr. Grunsky.

The low-water flow of streams entering Gatun Lake during four dry months will average about	1 200	sec-ft.
The water obtainable from storage in Gatun Lake, 5 ft. (87 to 82 ft.), for the same period, will amount to	2 210	"
The rain on the lake's surface, about 2.60 in. during four dry months, will be	100	"
Total	3 510	sec-ft.

There will be required:

To supply the evaporation loss, 0.0215 ft. in depth per day, from 164 sq. miles	1 140	sec-ft.
To supply leakage at structures	250	"
For power, and other uses	200	"
Total	1 590	sec-ft.

There is, of course, some uncertainty relating to the loss of water by evaporation. Experiments made at Boston, Mass., and at Kingsburg, Cal.,* indicate that, for a mean monthly temperature of 78.5° Fahr., which is about that which will prevail during the dry months at Gatun Lake, an evaporation of 0.0215 ft. per day may be expected from an open water surface. This is about 7% in excess of the evaporation rate assumed by General Abbot in 1905. Canal Commissioner H. H. Rousseau, Assoc. M. Am. Soc. C. E., in an address before the Trans-Mississippi Congress in August, 1909, places the evaporation loss at $\frac{1}{7}$ in. per day, but does not state how he arrives at this amount.

For a mean monthly temperature of 78.5°, the Boston curve indicates an evaporation rate of 0.021 ft. in 24 hours. The highest observed points on the Boston curve are for mean monthly temperatures of 72° and 70° in July and August, and corresponding relative humidities of about 71.5 and 75.5 per cent.

The Kingsburg curve indicates a rate of 0.022 ft. for a mean temperature of 78.5 degrees. The highest points on this curve, based on observation, are for mean monthly temperatures of 82° and 84°, in July and August, with corresponding relative humidities of 32 and 35 per cent.

On the Isthmus the mean monthly relative humidity during the dry months is about 84 per cent. It seems probable that the actual evaporation rate from a large body of water on the Isthmus, owing to the high moisture content of the atmosphere, will be somewhat less than the rate introduced into the calculation; but a satisfactory basis for a humidity correction is lacking.

* "Evaporation from the Salton Sea," by C. E. Grunsky, M. Am. Soc. C. E., *Engineering News*, Aug. 13th 1908, p. 163.

Mr. Grunsky.

The leakage at structures, above noted, is the estimate of the Board of Engineers of 1905. The requirement for power has also been introduced into the calculation at the Board's estimate. The present plans contemplate the use of much more water for power, but this is immaterial, as, in case of need, all available water would be used for lockages during the dry months and none at all for power, which can always be otherwise secured.

According to the above figures, on the assumption that the lake is full at the beginning of the dry months, there would be available for lockages and to supply the seepage and other indeterminate losses, 3 510 — 1 590 = 1 920 sec-ft. On the extreme assumption of no seepage loss and no allowance for evaporation from marginal wet lands, all of the 1 920 sec-ft. would be available for lockages.

Following General Abbot's method of estimating the quantity of water required per lockage, on the assumption that a reduced lock length will be used for 80% of the lockages, the average requirement per lockage per day through the Canal would be about 55 sec-ft. According to Mr. Sorzano, each lockage per day will require 96 sec-ft. It is probable, of course, that the Canal will be used for some time as assumed by General Abbot, but the possibility must also be admitted of a demand on the Canal to its capacity within comparatively few years. This capacity may be limited by the number of lockages that can be made per day, or by the number for which water is available. The maximum traffic, expressed in tonnage and not by the number of lockages, will undoubtedly be attained when the locks are always crowded full of shipping and no lockages are made with intermediate gates. By the method of estimating used by the Board of Consulting Engineers and by General Abbot, there should be required for each such lockage through the canal 6 900 000 cu. ft. of water, or 80 sec-ft. for 24 hours.

The available water, therefore, would permit of 35 lockages per day, if 80% are lockages with intermediate gates. On the author's assumption of the requirement per lockage through the canal, there would be water for 20 lockages per day. As the Board of Engineers would estimate (intermediate gates not used), there should be water enough for 24 lockages per day when the demands of commerce have reached the canal capacity. If no water were used for power, these figures might be increased by 4, 2.5, and 2, respectively.

The maximum traffic through the Canal, as limited by the speed with which vessels can be passed through the locks, is given at 40 per day by Alfred Noble, Past-President, Am. Soc. C. E., and Joseph Ripley,* M. Am. Soc. C. E. It is 48 according to Mr. Rousseau,† who probably voices the opinion of the Canal authorities. Messrs. Noble

* Report to the Board of Consulting Engineers of 1905.

† Address at Denver, in August, 1909, before the Trans-Mississippi Congress.

Mr. Grunsky. and Ripley place the extreme number at 57, but state that, after allowing for all causes of delay, it should not be hoped to realize more than 40.

These numbers are far in excess of the 20 to 24 lockages estimated from the water supply. The water supply, in other words, does not appear to be sufficient to bring the canal up to the tonnage capacity of the locks without utilizing more storage than is now contemplated. Each additional foot below the 82-ft. level drawn from Gatun Lake will add about 4.5 to 5.5 lockages per day during a period of 120 days, this number depending on whether the net contents of the locks, or the figures suggested by Mr. Sorzano, should be introduced into the calculation.

Any seepage losses from Gatun Lake will reduce the number of lockages by one per day for each 80 to 96 sec-ft. of such loss. That there will be some such loss seems unquestionable. If it should approximate the amount suggested by the author—about 1 960 sec-ft.—it would indeed be a serious matter. The writer sees no cause for alarm. He believes that this loss will be small, and that it should only be considered as one of the elements of risk which, in some measure, may reduce the efficiency of the lock canal. The distance which must be traversed by percolating waters from Gatun Lake to any outfall point, except near the various dams, is so great that the velocity of motion will be very slow. The measured losses of water from canals, as cited by Mr. Sorzano, can only be used as evidence that seepage losses are possible; they afford no criterion of the amount of such loss from a lake of large extent. The measurements in the case of the Kings River and Fresno Canal, referred to in the paper, were made by the speaker. The canal lies in a flat alluvial territory, and the seepage was but a few feet vertically downward to the water-table.

The available data indicate that the traffic capacity of the Panama Canal, during the dry period, will be limited by, and is to be determined from, the available water supply. The best method of holding the water supply throughout the year at the capacity of the locks is, therefore, a matter which should receive timely attention.

Mr. Hodges. H. F. Hodges,* M. Am. Soc. C. E. (by letter).—Mr. Sorzano's paper on the probable water supply for the lock canal at Panama is most interesting. The conclusions he reaches are so different from those of others who have reasoned from the same data, that the writer feels impelled to join in the discussion which he so kindly invites.

The author's remarks on the requirements of navigation admit of no convincing contradiction. Anyone is at liberty to prophesy on this subject. The dimensions which are now fixed for the locks of the Panama Canal, and the depth which has been given to its channels,

* Lt.-Col., Corps of Engineers, U. S. Army.

express the judgment of the authorities responsible for these features as to the probable future needs of navigation. If the draft of vessels desiring to use the Canal should grow beyond its depth, however, it may be said that, not only the Canal, but the great majority of harbors now available, would have to be deepened in order to accommodate these monster vessels; and that the cost of deepening the Canal and renewing its locks would be only a small part of the expense to which mankind would be forced by such a development. Mr. Hodges.

The clearance which Mr. Sorzano regards as necessary (6 ft. below the keel in a reasonably good channel), seems a little large, and his requirement of 3 ft. below the keel in a lock is unquestionably more than is needed. Large vessels pass through the St. Marys Falls Canal drawing practically every inch of water which the elevation of the sill will permit, and sometimes, strangely enough, a little more.

Table 1 is possibly a little misleading in its reference to the actual and the assumed dry season. It is true that the actual number of rainless months on the Isthmus sometimes exceeds the ordinarily assumed length of the dry season, but as far as its effect on the water supply of Lake Gatun is concerned, this is not important. The rivers continue to run with much more than their ordinary dry-season flow for some time after the rains have ceased, so that if a rainless season of 5 months is assumed, there would only be a season of 4 months of minimum water supply in the lake. These periods, of course, are given in round figures, but they will justify Mr. Sorzano in assuming in his computations a dry season of only 120 days, although indicating one of more than 150 days in Table 1. Mr. Sorzano, however, has overstated his case in saying that "the average discharge of all the streams that can empty into the proposed high-level lake at Gatun, will rarely exceed 1 100 cu. ft. per sec. in dry years." The minimum daily average for the 4 months of the dry season, namely, the average in the worst known dry season for 19 years, is 1 190 cu. ft. per sec., and the average dry season has a daily average discharge of 3 030 cu. ft. per sec. For the 3 driest months, the discharge is less, but the quantity stored in the lake, if distributed over 3 months instead of 4, would more than make up the difference.

It is in its assumptions as to the consumption of water that the paper differs most from discussions which have led to the usual belief in the adequacy of the water supply. The assumption as to surface evaporation is somewhat greater than that which experience on the Isthmus seems to justify. The average evaporation for the dry months, shown by observations, is 6.6 in. per month for the 3 driest months, and 6.026 in. when the dry season is considered 4 months long. In the remaining months of the year it is approximately one-half of this. Mr. Sorzano's assumption of $\frac{1}{4}$ in. or more per day is somewhat in

Mr. Hodges. excess of the actual determined amount. The writer, however, believes that a credit to the water supply of considerable moment has been overlooked. The contribution to the waters of the lake during the dry season is the discharge of the Chagres River and its tributary streams. Its stated amount has been estimated, not from the rainfall and catchment area, but from the measured discharge of the rivers in the dry months. During these months, as well as during the rainy months, the area which will eventually lie under the water of the lake is now covered by a dense growth of lush vegetation, which, even in the dry season, draws up and gives off to the air a tremendous amount of moisture which never appears in the measured discharge of the rivers. It is known that the amount so given off by growing vegetation may reach approximately the figures assumed for the evaporation. This loss will cease when that from evaporation comes in. In debiting the lake with evaporation from its surface, one should not forget to credit it with the unknown amount by which the river flow is now reduced through the medium of vegetation growing over the lake bed. No fixed value has been attributed to this amount in any discussion of the lake supply. It has been held simply as an indefinite although considerable reserve. Mr. Sorzano's evident respect for border evaporation, or "fly-off," indicates that he will probably be sensible of the importance to be attached to this reserve.

In connection with border evaporation, Mr. Sorzano states that the drying up of the lake "in every dry season, will uncover an area of land measuring more than 100 sq. miles." This, of course, depends on how much of it dries up. The area of the lake, at + 87, is 167.4 sq. miles, and at + 80, is 153.3 sq. miles. The writer's studies lead him to believe that it will not dry up even as far as the latter reference, and if it does, Mr. Sorzano's area of 100 sq. miles will be reduced to less than 14 sq. miles. In the average dry season the writer believes that the lake will not shrink noticeably.

As for the percolation through the bottom and sides of the reservoir, the author admits that it can only be guessed at, and the writer gives cordial assent to that proposition. Efforts to determine it, however, have not been lacking. The gentleman whom Mr. Sorzano compliments as being a very able engineer, and from whom he quotes as to certain experiments for the foundations of the dam at Gatun, has also made quite extensive borings at different portions of the rim of the reservoir area and of its bed, in places where the configuration and geological formation of the soil indicated that such investigations were most necessary. The result of these investigations has not shown any area which can be regarded as dangerous, and has strengthened the confidence of the engineers on the Isthmus that the percolation through the bed and walls of the reservoir will not be a matter of serious importance.

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In reference to this matter, the fact is worthy of notice that the Camacho Reservoir, to which Mr. Sorzano alludes as a horrible example of the permeability of Isthmian rock, supplies, from a water-shed of about 1 sq. mile, a total of about 2 000 000 gal. daily. This amounts to a flow of 3.1 cu. ft. per sec., aside from all losses due to evaporation, percolation, "fly-off," and what-not. Such a flow over the water-shed tributary to Lake Gatun, after deduction for the losses mentioned, would leave about 4 000 cu. ft. per sec. available, an amount greatly in excess of any probable need.

The author's discussion of the consumption of water in lockage requires a little correction. Owing to the fact that the upper lock in the Gatun and Miraflores flights is, for certain reasons, somewhat larger in area than the lower ones, the lifts between the upper and lower pools at these localities are not divided equally among the locks; and, at Gatun, the normal lift of the upper 1 000-ft. lock will be 26.38 ft., when the lake is at + 85 and the sea below at mean level. The lift of the other two 1 000-ft. locks at Gatun will be 29.31 ft. each, at the same water stages. At Pedro Miguel the normal lift will be 30 ft. at lake levels of 85 and 55, and, at Miraflores, the normal lift of the upper 1 000-ft. lock, with the lake at + 55 and the sea at mean tide, will be 26.10 ft., while the lift of the lower lock will be 28.90 ft. These figures will alter somewhat the lockage prisms assumed by the author.

Neglecting losses by leakage, etc., the total draft on the summit level, due to the passage of a vessel through the Canal, should be two prisms of lift and no more. These two prisms are those in the upper locks at Gatun and at Miraflores, and since, for reasons which will appear later, the opportunity of saving water at Pedro Miguel is so much greater than at Miraflores, the latter place will rule the water consumption. The displacement of the vessel, although mentioned by Mr. Sorzano, does not have any influence on the net result of a complete passage. It increases the consumption for a vessel going up and decreases that for one on its downward lockage. Neither of the two prisms of lift which each passage takes from the lake, however, must, of necessity, be a full prism; and that leads to an investigation of the statement, inferentially accepted by Mr. Sorzano, that intermediate gates cannot be used to advantage in the three-lift flight at Gatun. This statement crept into the Report of the Board of Consulting Engineers of 1905, and, apparently, has received circulation and acceptance since. The writer craves indulgence for an endeavor to prove its fallacy.

In considering the operation of the twin flight of locks, it should be remembered that it is intended to reserve one flight for up-bound and one flight for down-bound vessels. The water in each lock of the up-bound flight will regularly be left at its high level, and in the down-

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bound flight, regularly at its low level. Let Fig. 1 represent a three-lift flight of locks, each divided, as are those proposed for Gatun, into two unequal parts by intermediate gates, *n-e*, *d-b*, and *l-m*. Let the chambers, A, A_1, and A_2, be equal in area, and call this area A, and represent the area of the lock, B_1, by B. Suppose a vessel to have passed down, using the 550-ft. locks; then, since it must pass from lock A_1 to A_2, through B_1, and the gate, *g-r*, must then be shut to lower the boat to sea level, it follows that the level in B_1 as well as in A_1, after the downward passage of the vessel, will be the high level of the lock A_2. Represent the lift between the high levels of successive subdivisions, as follows:

$$B \text{ to } A = o;$$
$$A \text{ to } A_1 = x;$$
$$A_1 \text{ to } B_1 = y;$$
$$B_1 \text{ to } A_2 = z;$$
$$A_2 \text{ to sea} = w.$$

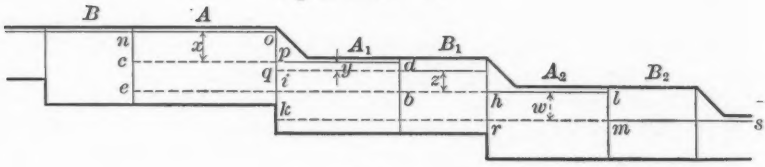

FIG. 1.

Fill A to lake level, *n-o*, and admit the vessel; close the barrier *n-e*. Then, since the level in A_1 and B_1 is the high level of A_2, represented by the line, *e-l*, the total quantity of water above that level, *e-l*, in the two locks, A and A_1, before equalizing, is represented by the prism, *n-e-i-o*. If, now, keeping the gates and valves in the barriers, *n-e* and *d-b*, closed, the valves in the barrier, *o-i*, be opened, the water level, *n-o*, in A, and, *i-b*, in A_1, will equalize at *p-d*. The prism, *c-e-b-d*, which is now above *e-l*, must be equal to the prism, *n-e-i-o*, which was above that level before equalizing. From this we have, since *o-p* is equal to x, *p-q* to y, and *q-i* to z, and the areas of A and A_1 are equal:

$$A\ (x + y + z) = 2\ A\ (y + z);\ \text{hence, } x = y + z \ldots\ldots\ldots (1)$$

Since the barrier at *d-b* was kept closed, the level in B_1 during this operation has remained at *b-h*. Let the vessel now be passed into the lock, A_1, and the gate and valves in *o-k* be closed. In A_1 and B_1 the total prism above *b-h* is *p-i-b-d*, equal to $A\ (z + y)$. Open the valves in *d-b* and the water levels in A_1 and B_1 will equalize at some level, *q-g*, the total prism above *b-h* now being *q-i-h-g*, equal to $(A + B)\ z$. Equating these two prisms, we have:

$$A\ (y + z) = (A + B)\ z;\ \text{hence } z = \frac{A\ y}{B} \ldots\ldots\ldots\ldots\ldots (2)$$

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Now open gate, *d-b*, and the valves in *g-r*, and equalize the levels between the combined chamber A_1-B_1, and the chamber A_2 in which the water level is at the line, *r-m*. Equalization will take place at some level, *h-l*, at a distance, *w*, above sea level, and, by the reasoning already followed, we have the prism, *q-k-r-g*, equal to the prism, *i-k-m-l*, or,

$$(A + B)\ (z + w) = (2A + B)\ w;\ \text{hence}\ A\ w = z\ (A + B)\ldots(3)$$

Combining Equations 2 and 3, we have $w = \frac{y}{B}(A + B) = \frac{A\ y}{B} + y.$

Combining Equations 1 and 2, we also have $x = \frac{A\ y}{B} + y$; hence $x = w$. We find, therefore, that *x*, which we know to be equal to $y + z$, is also equal to *w*, and consequently, that the lifts between consecutive 550-ft. locks are the same. Now admitting the vessel to the chamber A_2 and closing the barrier *g-r*, the water in A_2 can be lowered to sea level through the valves in *l-m*, and the vessel passed out through B_2 with the total loss of the prism, *h-r-m-l*, which is $A\ w$, and equal to the prism, $A\ x$, originally taken from the upper level. The levels in the flight will be left as found.

In order to accomplish the result it is necessary only that after the vessel has been admitted from the combined lock chamber, $A_1 + B_1$, to the 550-ft. lock chamber, A_2, closed below by the barrier *l-m*, the barrier *g-r* should be closed behind it before it is lowered from the level, *h-l*, to sea level. In this way the vessel drawing full draft can be passed through the flight with only the expenditure of water due to the prism of area *A*, or the 550-ft. lock, and the height, *x*, or $y + z$, or *w*, equal to one-third of *A*. The lock, B_1, becomes practically a portion of an intermediate canal, the water level never going below the normal high level in A_2, or *h-l*. There is no necessity for making two operations, as indicated above, of lowering the vessel from *p-d* to *h-l*. The valves in *d-b* and *g-r* may be opened at once, allowing the prisms in A_1 and B_1 to flow simultaneously into A_2, thus avoiding a slight loss of time.

If one takes the up-bound instead of the down-bound flight, the locks will regularly be found full, but the reversal of the operation will lift a vessel of suitable size from the low level to the high level with only the expenditure of one 550-ft. prism of lift, and without necessitating diminution in draft or delay in transit. A similar discussion will show that even the smaller 350-ft. lock may be used, with a very small expenditure of water, in locking small vessels through. The operation is somewhat more complicated, as the lock flight is divided into more lifts in order to accomplish this end, but the lifts are lessened and the time of each lockage, therefore, is made shorter; so that, while the lockages would be increased in number, it is doubtful if any increase in the time of passage would result. The saving of water through the use of the smaller locks is continuous as long as the series

Mr. Hodges. of small lockages remains uninterrupted. Under certain circumstances, the interruption of the series by a vessel requiring the full size and full depth nullifies the saving due to one of the previous lockages.

The writer feels that he owes an apology for the length of the foregoing demonstration, but he has thought it necessary in view of the general acceptance of the statement that intermediate gates cannot be used to advantage in the three-lift flight.

There is also another means of saving lockage water, of which Mr. Sorzano had probably no information. The middle walls of the locks are designed so that communication, controlled by valves, can be opened between the twin-lock chambers, and one can be used as a reservoir for a portion of the prism of lift in the other. A lockage in one direction may then require only a portion of the normal prism, leaving the remainder stored up for use in the upper twin lock of the other flight. Such saving in case of the flights is possible only with vessels which do not require the full draft of the normal prism over the sills, but it is very evident that the great majority of vessels will be of this description, especially those which can pass through the smaller locks. The possible saving due to this cross-storage of water is greatest at Pedro Miguel, where there will be no loss of depth by cross-filling, and where, under the most favorable circumstances, 50% of the lockage water can be saved, and under ordinary circumstances a quantity varying between 25 and 50 per cent. It is less at Gatun where the maximum possible saving is about 24%; and it seems safe to count on about 15 per cent. At Miraflores, the saving should be intermediate between that at Gatun and at Pedro Miguel. The existence of this possible saving is the ground for the previous statement that the expenditure of water will be ruled by Gatun and Miraflores, rather than by Gatun and Pedro Miguel.

Mr. Sorzano may be correct in his opinion that the development of commerce will lead to the use of the large locks with a number of vessels, instead of the small locks with one or two ships. If this be the case, each lockage will mean several passages instead of one or two, and the net result to commerce will be approximately the same. Each one of the fleet lockages will take a longer time than the one involving a single vessel, and, consequently, there will be fewer of them in the course of the 24 hours.

It is difficult to foretell just what line of development commerce will take. When the Poe Lock at St. Marys Falls Canal was designed, its dimensions were based on the methods of Lake carriage then existing, with one steam barge and a number of tow barges, it being intended to take in, as far as possible, one entire tow, two ships abreast. Methods of carriage on the Upper Lakes have changed since then, the large single vessel having come to the front; and if the lock were to be

built now, less attention would be given to taking in a fleet of relatively small vessels two abreast, and more to taking in two large vessels tandem. The canal on the American side at "The Soo" now averages approximately one and one-half boats per lockage. Mr. Hodges.

The author's intimation that the use of the intermediate gates will be objectionable on account of the great number of safety appliances to be used for each vessel at each lock, and the necessity of moving all vessels at extremely slow speed, can apply only to the use of the 350-ft. lock. No more movements are necessary in locking with the 550-ft. lock than with the 1 000-ft. lock. As stated before, however, when the small 350-ft. lock is in action, the method of operation may be such as to increase the number of lifts in order to save the greatest possible quantity of water. Nevertheless, it is not necessary that the number of lifts should be increased, even with this small lock, unless full draft is needed, and, even should it be thus increased, the lifts themselves are correspondingly diminished, the time of each lockage is correspondingly reduced, and, furthermore, the small size of the vessel will render its movement from lock to lock more easy and rapid. The writer is not inclined to accept, without reserve, the general statement that the use of the intermediate gates will be attended by delay, believing that the reverse will be the case.

The author's estimate of the water used for hydro-electric plants indicates that he has been misled by the knowledge that the Commission is now building two generating plants at Gatun and Miraflores. Apparently, he has assumed that it is intended to generate the full power at these two localities, and, therefore, he has stated that "it is intended to use about 6 000 h.p."

The present intention is to develop for operation somewhat less than 2 000 h.p., and this at Gatun. Ordinarily, the Miraflores plant is intended to lie idle, and only a part of the Gatun plant is to be in use. During the period of construction it is necessary to have the two plants, and, using steam, to develop a power considerably greater than is contemplated for the operation of the Canal in after years, when the power will be developed by water. The calculations which have been made on the Isthmus as to the water supply assume an expenditure of 275 cu. ft. per sec. for the permanent development. After the Gatun plant begins to supply the operating needs, the Miraflores plant, which will still have auxiliary boiler capacity, will be held in reserve ready to be thrown into the line in case of any failure. Having boilers installed, steam can be used should it be found desirable for any reason, in which case the estimated expenditure of 275 cu. ft. per sec. from the lake will cease.

In the author's discussion of the losses due to gates and valves, he has been rather hard on the Canal. It is true that the Gatun locks

Mr. Hodges. will have 40 gate leaves and a great number of valves, but there are only 4 gate leaves and 3 sets of culvert valves there through which the lake will lose water. The others will have their own leakage supplied by the leakage from above. If it be assumed that there is an opening $\frac{1}{32}$ in. wide around the meeting faces of the effective gates and valves, the leakage at Gatun will not reach half the amount mentioned by Mr. Sorzano. The assumptions made on the Isthmus as to the leakage through gates and valves, including the spillway, give a combined total of 275 cu. ft. per sec. for both ends of the upper level.

Any conclusions as to the water to be wasted in maintenance operations (Cause VII in the paper), must be largely guesswork. Undoubtedly, however, most of the intentional operations which will involve waste of water can be made to occur during the rainy season, when the loss is of no consequence. The writer cannot possibly accept the author's claim that 125 cu. ft. per sec. must be chargeable to these operations, including the power plants and hydraulic motors operating on Lake Gatun. If these plants and motors are private property, it will be a very simple matter for the Government to stop any draft of water should it be found desirable. It is not part of the duty of the Canal to supply them. If, on the other hand, the author refers to the Government plant, that has been reckoned in elsewhere.

The author's statement that the safety appliances and towing devices, now arranged for double-track service, will make single-track operation very dangerous, does not appear to the writer to be justified. Single-track operation of a flight of locks is undoubtedly less convenient than double-track operation, but the safety appliances and towing devices, to which the author alludes, introduce into the single-track operation no element of danger not present in the double-track operation. The twin flights of locks are intended to be entirely independent of each other in this regard.

The author's Cause IX, "Losses due to False Maneuvers and Accidents," is also a matter of speculation. He mentions, however, the 1 104 hours of stoppage due to breakage at the St. Marys Falls Canal, Mich., during its 2 007 days of operation, as an indication of the amount of water loss which may be expected in the Panama Canal. There is no statement of how much water loss at "The Soo" was involved by these hours of delay. From the best of the writer's information, he is inclined to believe that there was considerable time lost, but very little water.

The losses at the spillways and regulating gates of the Panama Canal can be controlled to a certain extent. The discharge over the spillway is to be regulated by crest gates about 19 ft. high, under the lower edge of which the flow will take place. When these gates are closed, it does not seem to be impracticable to pack the vertical edges

and sill in such way that there will be little, if any, leakage, even under the 16-ft. head to which they will be exposed. Mr. Hodges.

The author's Cause X, "Miscellaneous Losses," also seems to the writer to be somewhat overdrawn. Among these losses he includes all the needs of the population and industries which may locate around the borders of the lake and on its numerous islands. The population of the country during the construction of the Canal is very much greater than it was before, and the needs of this increased population are now adequately supplied by reservoirs which lie either above the level of Lake Gatun or are not tributary to it at all, and the waters impounded in these reservoirs are not counted in the estimated lake supply. If these reservoirs prove insufficient for future consumption, they can be supplemented by others which will store wet-season water for dry-season use, without affecting the dry-season supply for Lake Gatun. Mr. Sorzano also mentions the consumption and replenishing of all passing vessels and craft using the Canal. If this prove a source of inconvenience, the vessels can be made to get their water supply elsewhere, as they do now, and as they would have to, were the Canal to be built at sea level.

He also mentions the losses and waste at the intermediate section from Pedro Miguel to the sea-level section at Miraflores, due to evaporation, percolation, "fly-off," etc. Of course, there will be evaporation from the small lake at Miraflores, but it also has certain small independent sources of supply which are greater in proportion to its area than those of Lake Gatun. It need not draw on the latter to supply its losses by evaporation, etc.

Assuming the area of Lake Gatun at 164 sq. miles, and the depth of storage available, as stated by Mr. Sorzano, at 7 ft., namely, from + 80 to + 87, the total volume of water thus stored is equivalent to a flow of 3 090 cu. ft. per sec., for 120 days. Adding this to the minimum supply of 1 190 cu. ft. per sec. gives a total supply of 4 280 cu. ft. per sec. available for all uses during the dry season of 4 months, without drawing the lake down below + 80.

The observed rate of evaporation during a dry season of 4 months is equivalent to a flow of 930 cu. ft. per sec. for the lake area. In his calculations, the writer has assumed the leakage at the gates and valves at 275 cu. ft. per sec.; seepage, waste, etc., at 85 cu. ft. per sec.; and lighting, power, etc., at 275 cu. ft. per sec. In this assumption, as well as in Mr. Sorzano's, the credit due to saving in the lake the quantity of water which is now poured out by the vegetation covering its bed has been neglected, and the element of percolation in the bed, which is entirely unknown, has been omitted. The quantity of water required for lockage has been obtained by the following assumptions.

Mr. Hodges. which may or may not prove to be justified in fact. The writer gives them for what they are worth.

TABLE 8.

(1) Lock.	(2) Area.	(3) Normal Lift. Gatun.	(3) Normal Lift. Miraflores.*	(4) Total water for complete passage, in cubic feet.	(5) Column 4: in cubic feet per second.	(6) Weight.	(7) Column 5 weighted.	(8) Saving due to cross-filling.	(9) Column 7 corrected for Column 8.
1 000 ft.....	133 800	26.38	26 10	7 022 000	81.5	5%	4.08	16%	3.42
900 ft.....	122 800	28.10	27.40	6 810 000	78.9	15%	11.83	16%	9.93
550 ft.....	82 400	28.33	32.75	5 033 000	58.3	50%	29.15	15%	24.77
350 ft.....	40 400	18.58	30.16	1 969 000	22.8	30%	6.84	11%	6.08
Totals	.....	.....	.....	.....	.....	.....	51.90	....	44.2

* No intermediate gates in lower lock at Miraflores.

The plan of the upper lock is such that lengths of 1 000 ft., 900 ft., 550 ft., or 350 ft. can be used, the 550-ft. and 350-ft. lengths being capable, by squeezing, of increase to about 600 ft. and 400 ft., respectively. In preparing Table 8, the assumption has been made that 5% of the vessels arriving would require the 1 000-ft. lock; that 15% would require the 900-ft. lock; that 50% will pass in the 550-ft. lock, and the remaining 30% in the 350-ft. lock. With these assumptions, and taking into consideration the saving in water which can be effected by cross-filling, the total average consumption for one passage per day, *i. e.*, for two prisms of lift each day, is 44.2 cu. ft. per sec. Neglecting any advantage to be derived from cross-filling, it is 51.9 cu. ft. per sec. Deducting the sources of loss assumed above from the total available supply would give 4 280 — 1 535 = 2 745 cu. ft. per sec., which, at the calculated value for cross-filled lockage, will allow 48 complete passages of the Isthmus per day with a remainder of 623 cu. ft. per sec. as a margin. The writer believes this margin to be sufficient to cover underestimate of losses, and feels confident that the water supply will prove adequate to the needs of the Canal for a great many years, if not for all time to come. Should it ever become necessary to increase it, this can be done very simply by the construction of a dam on the Upper Chagres River, at Alhajuela or elsewhere, which will permit of increased storage to supply the deficiency during the dry season. The duty of building this dam may be properly left to posterity, should the need ever arise.

The foregoing estimate of the water supply is based on the minimum observed daily average for the 4 dry months, *viz.*, 1 190 cu. ft. per sec. The daily average supply for the average dry season of 4 months is much greater, being 3 030 cu. ft. per sec. In the average year, therefore, the surplus will be greatly increased.

In the foregoing, as in Mr. Sorzano's paper, a 7-ft. storage, from + 80 to + 87 in Lake Gatun, has been assumed. In the recent plans, the

storage has usually been taken at only 5 ft., from + 82 to + 87, and even with this reduced variation in level, it is thought that the supply is sufficient for all lockages through a dry season of minimum flow. In other words, instead of the lake level being reduced, as concluded by Mr. Sorzano, it will not be necessary to bring it below + 82, giving a depth of 42 ft. in the Culebra Cut, under the most unfavorable circumstances. For many years it may be held desirable to operate at a lower level, until vessels requiring the full draft desire to use this route; but, in the writer's judgment, there will be no necessity, due to scarcity of water, for such low-level operation. Mr. Hodges.

THEODORE PASCHKE, M. AM. SOC. C. E. (by letter).—The author has treated his subject most ably. His conclusions are startling, and if, after a full discussion, they still prove to be approximately correct, they are a severe arraignment of the designs for this great work. Mr. Paschke.

Of the various items affecting the water supply of the Canal, Cause III, "Percolation through the bottom nd sides of the reservoir," has attracted the writer's particular attention, as it embraces a feature which has not been mentioned in the paper, and, apparently, has been overlooked by Mr. Sorzano.

The author's deduction that the water level of the lake will suffer a certain lowering during the 4 months of dry season, by percolation or seepage through the bottom and sides, is based on certain assumptions of the existence of fissures and seams in the rock formations and the general porosity of the subsoils in the locality, all of which in turn have been the subject of close scrutiny, investigation, tests, and other researches by the Canal engineers. These assumptions, therefore, are fair and are not to be gainsaid.

For the sake of argument, it is also admitted that the deductions therefrom—the amount of seepage—will conform to the conditions existing when the reservoir is first being filled. From that time, sedimentation will take place all over the bottom and sides of the lake, and attention is called to this feature. If the writer is not misinformed, the soils which overlie the rock formation of the Isthmus are composed mainly of volcanic deposits. These soils yield easily to the erosive action of running water resulting from the torrential downpours peculiar to the rainy season in the tropics.

In a stream like the Chagres, having its origin in a mountainous region overlaid by such volcanic deposits, the flood waters are charged to their capacity with the eroded particles of the soil. The finer particles, held in suspension, are never deposited along the sides or bottom of the stream, but, impelled by the current, are carried out to sea to furnish material for new combinations and formations in Nature's great workshop. In cases where the flood waters overflow the banks of the stream into adjoining and comparatively level regions, where the flow of the waters is arrested, the suspended matter is slowly

Mr. Paschke. deposited, thus forming, in the course of time, a well-nigh impervious layer over these stretches, constituting the basis for the swampy character of regions which otherwise, due to their elevations, should drain themselves.

Only the coarser and washed particles carried by the flood waters are found deposited in old and new river beds. The opinion is here ventured that the low hills of the Gatun region consist of such washed-out material deposited from time to time by the flood waters of the Chagres. This would explain the high seepage flow through some of the Gatun soils given in Table 3.

By throwing the artificial barrier across the Chagres Valley at Gatun, the waters of that river will be impounded and form a lake. The effect will be that all the sediment brought down by the flood waters during the rainy season will be deposited in this lake. The coarser and heavier particles will be deposited at the point where the flood waters come in contact with the lake water, and the lighter and finer particles will be carried a little farther before they settle to the bottom. The lightest and finest particles, however (the volcanic dust particles in combination with organic matter held in suspension), will be diffused throughout the waters of the lake, reaching every nook and corner, where they will gradually settle to the bottom. This process of settlement of the finer particles will go on with more or less uniformity throughout the rainy season, which lasts about 8 months of the year.

It is not difficult to foresee the final result, provided the Gatun Dam stands and does not slip and disintegrate (as predicted by some), or is not washed out on account of some ramming accident at the locks (as predicted by others).

Granted that the soils forming the sides and bottom of the proposed lake are porous and that the underlying rocks are shattered, full of seams and joints, and that for these reasons the lake bed, at the beginning, will be nothing more than an immense filtering bed, surely the sedimentary deposits from the Chagres and other inflowing streams will eventually choke up the filtering features and ultimately seal effectively every pore, seam, and joint in the material forming the bed. Indeed this whole process is nothing more than a gradual filling up of the lake with the spreading of a water-proof blanket over the bottom and sides in advance of the coarser filling material.

The views here advanced are not born of a desire merely to criticize, but represent the conclusions reached during a professional experience of 12 years in one of the Central American States, during which the writer had ample opportunity for observing similar phenomena in reservoirs, lakes, and rivers.

In conclusion it should be stated that these views should not be taken as a defense of the present designs of the great work. Any possible modification of the author's calculations, due to any tendency to retard and check the percolation of water from the lake, will be so

slight as not to affect in the main his general conclusions, which, in the writer's opinion, stand unimpaired. They are an appeal not only to engineers, but also to the whole American people, to pause and think a moment over the possibility of having the traditional national reputation for sagacity come to naught by any short-sightedness displayed in the persistence of the present designs of a lock canal. Mr. Paschke.

ALLEN HAZEN, M. AM. SOC. C. E. (by letter).—The writer will discuss only one feature of this paper, namely, the loss of water by percolation. The author's assumptions as to percolation differ radically from those which the writer has made, and the difference represents the greatest single difference between his calculations and the hydraulic calculations which were used in making the Report of the Consulting Engineers in February, 1909, to which the author refers. Mr. Hazen.

Under the heading "Seepage Velocity in Gatun Soils," reference is made to certain experiments by C. M. Saville, M. Am. Soc. C. E., Assistant Engineer of the Commission. From this report the author has derived conclusions which differ from the writer's, and it seems best to take up these experiments in somewhat greater detail. The experiments on soils are described on page 138, *et seq.*, of the Report of the Isthmian Canal Commission for 1908.

The soils were placed in galvanized-iron tanks, 39.36 in. in diameter, and 54.5 in. high. The tanks were coated inside with pitch and coarse sand. In the center of the bottom a connection was made which was protected by a piece of wire screening. Two ¾-in. pipes, with perforated cross-pieces, were carried through the sides of the tanks and embedded in the material to be tested. These were connected on the outside with pressure gauges in which the differences in head were observed. The water passing was collected and measured. As far as possible, an effort was made to deposit the material in the tanks in a condition similar to that when found in place. Every effort was made to drive out all the air from the material, and a constant head of water was maintained.

The discharges under conditions when the hydraulic slope equaled 1, or, in other words, when the head available to make water pass through was equal to the depth of the material, for the five materials tested, were as follows:

	Cubic feet per acre per 24 hours.
Stiff blue clay from test pit down the Chagres River, north of the dam site	49.9
Red sandy soil from embankment slope of spillway hill	226.7
Medium fine sand from test pits down the Chagres River	206.7
Brown sandy clay from Gatun Island test pit, between surface and 13 ft. below	136.4
Blue sandy clay from Gatun Island test pit, from 15 to 37 ft. below surface	295.7

Mr. Hazen.

Mr. Saville did not reduce these to inches per day, but the writer has done this, with the following results, for the five materials, respectively:

0.014,
0.062,
0.056,
0.037,
0.081,

average, 0.050, equal to $\frac{1}{20}$ in. per day.

In applying these results to the probable percolation of water from Gatun Lake, allowance must be made for the probable difference in hydraulic slope. Certainly the average distance from the water in the lake through the ground to a point where it could discharge outside would be a considerable number of miles. For the purpose of this calculation, assume that it is only 1 mile, and that the discharge in all cases is at sea level. There will then be a head of 85 ft. acting in a distance of 1 mile, or a slope of 1.62% in place of 100 per cent. By a well-known law of percolation, the rate of seepage will be reduced in the same proportion as the slope, and the percolation to be expected will be $0.05 \times 0.0162 = 0.0008$ in. per day.

This rate of seepage is not to be reckoned necessarily on the whole area of the bottom of the lake. It applies instead to the smallest section of the pervious material through which the water is obliged to pass on its way from the lake to the sea. There is no means of determining what this smallest section would be; certainly it would be only a fraction of the actual area of the lake. It is needless to attempt to determine it, because even if the rate be applied to the whole area of the lake, it amounts to less than $\frac{1}{1000}$ in. per day, or to the one six-hundredth part of the amount assumed by the author.

Mr. Saville's experiments with rocks, made by cementing a piece of rock into a 6-in. pipe, represent more difficulty and perhaps less conclusive investigations. The results, for 1 ft. in thickness of stones, scaled from Plate 103, accompanying Mr. Saville's report, are given in Table 9.

Although these figures exceed considerably those found for the soils, no effort will be made to carry the calculation further, as it is obvious that the rates of seepage indicated are too small to be of practical significance.

The materials on the Isthmus, as the writer saw them in February, 1909, are generally of a ferruginous, clayey nature. That is, they are clays and shales, part of the alumina in which has been replaced with iron to the extent that they contain something like 10% of metallic iron. The substitution of iron for alumina changes the physical properties of the materials to such an extent that Northern experience cannot be applied safely, and their ability to stand up in stable embank-

ments must be based on actual experience with them rather than on considerations of Northern precedent. Mr. Hazen.

TABLE 9.

No.	Kind of stone.	Source.	Head, in feet.	Percolation, in cubic feet per square foot per hour.	Computed Percolation, in Inches per 24 Hours.	
					Head equals slope.	85 ft. head in a distance of 1 mile.
1	Blue sandstone	Lock site	4	2.0	144	2.3
2	Volcanic ash	Lock site	40	2.2	16	0.27
3	Volcanic tufa	Quarry	40	0	0	0
4	Dark conglomerate	Lock site	40	0.01	0.7	0.011
5	Brown conglomerate	Lock site	40	0.12	0.86	0.014
6	Volcanic ash	Lock site	40	0.21	1.50	0.024
7	Light blue-gray conglomerate	Lock site	40	0.28	2.0	0.032
8	Argillaceous sandstone	Spillway	40	0.40	2.9	0.047
9	Gray sandstone	Spillway	40	0.26	1.9	0.030
10	Light brown sandstone	Spillway	40	0.71	5.1	0.082

In the whole line of the work traversed by the Canal there is no considerable deposit of silicious sand or gravel. The so-called sandstone does not contain silicious sand particles, but, in appearance and in physical properties, is more like shale than the rock which we know as sandstone. This so-called sandstone, said to be water-bearing, does not permit percolation through its pores. There are occasional seams in it which carry limited quantities of water, in the same way that shales and mica schists and granites are fissured, and carry water in limited quantities.

All these materials, as seen by the writer in the various cuts on the work, and as tested by Mr. Saville in his laboratory, are substantially impervious.

Of course, it is true that the engineers did not have an opportunity to see every part of the lake bottom. That would have been impossible under existing conditions, and especially in a limited time, but the materials which they did see extending from one end of the lake to the other, and, in many places to its sides, were all of the same general class, and were all generally impervious, and there is every reason to believe that the materials throughout the whole lake are of the same general character.

There is no more reason to apprehend loss of water from Gatun Lake by percolation than there is to expect loss in the Ashokan Reservoir now building by the City of New York. No allowance for such percolation has been made for the Ashokan Reservoir, as experience with the Croton Reservoir, the Wachusett Reservoir, and a hundred smaller reservoirs, has not indicated the necessity for it.

Mr. Hazen.

The author speaks of some of the dams built for water-works' purposes on the Isthmus, and of the leakage from them. The writer visited several and noted the conditions of leakage. These dams were made by filling with the materials most conveniently available to a not very large earth dam section, and apparently without any very deep preparatory excavation of the underlying material. In all cases there was a certain amount of seepage at the bases of the dams, coming apparently through the material under the dam a short distance below the natural surface. In one case a slight indication of leakage through the lower part of the embankment was observed. In no case was there indication of leakage through the deeper underlying materials.

The writer is at a loss to account for this percolation through the material a short distance below the natural surface. His first thought was that it might be due to the spaces left by decayed roots, but observation in cuts showed that as a rule roots did not extend far into the soil, and the seepage was through a layer deeper than that to which roots ordinarily extend. The writer has been told that in very dry seasons there are shrinkage cracks, due to the continued extraction of moisture, by roots and otherwise by evaporation, from the clayey soil containing a large percentage of pores filled by moisture, and that these shrinkage cracks sometimes extend to considerable depths. It may be that these cracks when once formed do not completely heal in the ensuing rainy season, and that there may be seepage through them.

The seepage from these reservoirs used for water-supply purposes in no case seems to be enough to affect the stability of the dams, nor to take away to any considerable extent from the quantity of water available for water-supply purposes.

The writer is interested to note that Mr. Sorzano raises no question as to the loss of water by seepage through the Gatun Dam itself, and he believes that all parts of Gatun Lake formed by it will be equally tight.

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AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

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RIVER PROTECTION WORK ON THE KANSAS CITY SOUTHERN RAILWAY, NEAR BRADEN, OKLA.

Discussion.*

By Charles H. Miller, M. Am. Soc. C. E.

Mr. Miller. Charles H. Miller, M. Am. Soc. C. E. (by letter).—Having visited this work several times, and having become familiar, to some extent, with the David Neale system during the past three or four years, the writer has read with much interest Mr. Lahmer's detailed description.

When the writer began the investigation of this system several years ago, he was confronted with the facts that there were no accurate cost or location records covering the work which had been placed, some of it, as many as twelve years before; that often there was no way of telling whether or not the changes occurring in the vicinity were brought about by the influence of the dikes or through natural causes; that some of the dikes seemed to have served well the purpose for which they were constructed, while others were found to have been entirely flanked and dropped into the bed of the river; that is, the river got behind the dike and continued to scour away the bank, until, in some cases, the dike was left on the opposite or bar side of the river.

In most cases it developed that no repairs or additions were made after the work was first constructed, it being the contention of the inventor that after it once became covered with sediment, it would be practically permanent.

* This discussion (of the paper by J. A. Lahmer, M. Am. Soc. C. E., printed in *Proceedings* for October, 1909, and presented at the meeting of November 3d, 1909), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

Mr. Miller.

One point, however, seemed to come out prominently in most cases, namely, that the dikes possessed an inherent strength against being broken up piece by piece and carried off by the currents, and although entirely cut down by scour underneath and behind, and much bent and twisted, the main mass, especially that under the low-water level, remained, those parts above low water sustaining losses of one or more layers due to the decay of some of the materials composing them.

With this quality in view, this system was adopted, under the assumption that sufficient repairs could be made from time to time to hold the dikes in effective position, and, in the end, that a protection would be obtained at less cost than for a continuous system.

At Myrick, Mo., on the Missouri River, dikes have been in place for three years, and their position is being maintained, but this is much too short a period in which to obtain a reliable comparison between this and other systems. It has developed that an error was made in placing these dikes too far apart, and it has been necessary to build short pieces of bank revetment, both above and below them, for distances of approximately 100 ft., in order to prevent them from being cut down. The original cost of this work was $2.50 per lin ft. of bank; the repairs and additions placed amount to $1 per lin. ft., making a total of $3.50 per lin. ft.

This system does not contemplate any particular size or shape of dike, but fits the physical conditions as well as the class of materials to be found in the locality, and, naturally, there is a difference of opinion as to the most effective shape. The writer now uses a form which differs very much from that described by Mr. Lahmer. It is triangular or pyramidal in shape,* and the main part does not jut out into the stream as much as the long and narrow dike described in the paper. This triangular dike is now modified by discarding the large fascines for the foundation, and simply using a mattress, either of the solid or of the mud-cell type; it having been demonstrated clearly that the fascine method is the more expensive, first as to its actual construction, and second, because it permits more scour under the edges, hence more settlement, and, consequently, more material is needed to construct the dike to the required height. The mattress foundation is constructed so as to extend out beyond the edges of the proposed dike, and, as the scour progresses, this extended portion of the mattress, being pliable, drops and prevents excessive underscour. The mattress, extending a short distance above and below the dike, also prevents the eddy which cannot be entirely overcome, from cutting behind the upper bank part of the dike.

In calculating the relative effectiveness of the different forms of dikes much depends on the height and duration of floods, but the writer is firmly convinced that for the Missouri River, where floods often

* Described in detail and illustrated in *Engineering News*, October 22d, 1908.

Mr. Miller.

remain at bank-full stage for a month or two at a time, the form of dike described by Mr. Lahmer will not hold up as well as it will in the Arkansas River, where the floods are usually of much shorter duration and that it will be necessary to place some top-bank protection both above and below the dike. In other words, the extent of the dike, up and down stream, at the top of the bank, will prove to be insufficient for a long period of high water. The dike described is intended to throw the main body of the river so far away from the main bank that the influence of any eddies which may develop, will not be felt at the bank. This will be found to be true for all ordinary stages of the river, at which times a considerable portion of the dike is above the water surface, and the water is forced around it at some distance from the general shore line or edge of the water. This distance gradually diminishes as the river rises, and when it is high, the water flows over the dike only a few feet out from the general shore line, and there is then nothing to prevent it from scouring out under the edge of the dike.

It is true that the stronger force of the currents is farther out and lower down from the top bank; but there is often sufficient force along the top bank during high water to cause much damage to such work. Mr. Lahmer calls attention to a caving, from 8 to 10 ft. in width, below Dike 5. It is just these small amounts at each recurrent flood which will soon result in the dike being on a narrow strip jutting out from the main bank, and this strip is usually quite susceptible to being cut out by eddy currents, thus allowing the upper part of the dike to settle down, thereby giving the currents freer play and a chance to scour more rapidly.

The necessity for extending the system of dikes or protection work such a great distance up the "bend" is not quite clear, in fact, it is believed that it would have been best not to have done so, at least during the first year. The best information at hand discloses no caving in this part of the river of an amount greater than about 300 ft. per annum; it is usually very much less; and, therefore, it is safe to predict that the railway tracks would have been perfectly safe for at least one year more, if no work had been placed above Dike 2, which is about 500 ft. from the track. Then, after the caving had continued so as to form a "point" in the "bend" at Dike 2, one or two additional dikes could be built each year, as the scouring progressed up stream, the net result being that new work would occupy the position of greatest attack as it moved up stream, and that the "false point" would tend to deflect the main river below the same so as to relieve the lower and older dikes. In fact, it is highly probable that it would deflect it so as to cause the sand bar to scour off to such an extent that the river would give no further trouble below Dike 8, and that by the time Dikes 2 to 8 became dilapidated from age they would no longer be needed.

Mr. Miller.

The form of the "bend," as now protected, is that which usually obtains in this part of the river, and it is quite likely to remain so for many years; which means that all these dikes must be maintained continuously, also that the tendency of the river will be to cave in gradually and straighten out below Dike 8, thus necessitating additional work to protect that part of the railway track.

If the plan to stop the upper end of the work with Dike 2 had been adopted, this dike, being in a very exposed position, would no doubt have been damaged considerably and possibly cut down entirely; however, it would have been much reinforced on account of its exposed position, and in all probability would not have been entirely lost. In any event, the track could not possibly have been reached, the desired "point" would have been formed, and the conditions for this particular case much improved. It is the almost positive assurance of a loss of some extent, when the head of the protection work is left within the limits of active caving, which is objected to so strongly by many, and is the reason for extending the work to a point above the caving, as was done in the work at Braden.

It is hardly safe to depend on the erosion noted on the north bank of the upper end of the large bar at the point marked "vertical bank" on Fig. 1, nor to assume that eventually it will extend down so as to bring the river below the dikes, thus obviating the necessity of maintaining them, because this would be a very unusual manner for this caving to progress. Had Fig. 1 been extended to show the "bend" above this point, it is quite likely that it would have disclosed a greater rate of erosion or caving at some distance above the "vertical bank," tending gradually to throw the impact of the river away from the "vertical bank," and also tending to keep the river directed toward the upper dikes in the protection system.

On the other hand, if this unusual state or condition did occur, it would be one under which the dike system would be extremely hard to maintain, the one under which the continuous system of revetment or protection would be the most economical in the first place, because it would be necessary no doubt to place continuous protection between the dikes as they came within the line of almost perpendicular attack.

The dike system does not give the same security, in fact, as the continuous system, and where the bank is as near the railway as is the case in the vicinity of Dikes 6 and 7, careful and constant watching will always be necessary during each flood period.

With the entire "bend" protected as it is now, no marked relief can be expected. In this case, however, the top bank is composed in large part of a hard clay, and, therefore, the progress of any caving which may develop when the river is high will necessarily be quite slow.

One reason for using the dike system is its economy as compared with the continuous system, brought about chiefly because the dikes

deflect the currents so that part of the bank needs no protection; and much greater economies can be brought about when advantage can be taken of a "point" in a "bend" so that it can be held as an enormous dike. Mr. Miller.

Mr. Lahmer describes the use of large bundles of brush or hollow fascines as temporary protection against the rapid erosion just before the general work was started, and again to stop the caving just below Dike 5 during the flood which took place after its completion. It is for such emergency cases that these large fascines are especially useful, the writer having on several occasions successfully stopped very rapid caving with them. These fascines can be built on the river bank, and launched into place without any floating or special plant of any kind.

Any variety of brush and poles can be used, and the size may be quite variable; if the brush contains no leaves and small branches, then weeds, hay, or straw must be used to form tight sides and partitions. It is also possible to use sawed lumber instead of brush and poles. Telegraph wire may be used to bind the bundles or fascines together, and several wires can be twisted together to form cables or strands for anchoring the fascines to the bank. Some spikes can be used to advantage also. In cases where the river does not carry much sediment, and, in any event, it is well to have some empty sacks to be filled with soil and placed within the fascines to cause them to sink into place more quickly.

All or a sufficient quantity of these materials can usually be obtained readily and rushed to the point of danger by a work train. Large gangs of laborers can build one or more fascines simultaneously, and roll them into the river as fast as they are constructed. The stage of the river is immaterial, as long as the area of the bank or dry land on which the fascines are to be built, is large enough.

It is earnestly hoped that Mr. Lahmer will present figures showing the cost of maintenance.